SCHNEIDER

SPECIFICATIONS FOR STRUCTURAL WORK OF BUILDINGS

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GENERAL SPECIFICATIONS FOR STRUCTURAL WORK OF BUILDINGS.

BY

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PREFACE.

This edition of the General Specifications for Structural Work of Buildings is a reprint from the one published in *Transactions* of the American Society of Civil Engineers, Vol. LIV, page 490 (1905), revised to date. It contains additional tables and other useful information, also specifications for concrete and reinforced concrete for building construction.

As reinforced concrete construction has lately come into extended use for building work, the writer thought it expedient to include a set of regulations covering its essential requirements, based on what he considers safe practice.

In preparing specifications for reinforced concrete, the writer has been guided by those already in existence, the most prominent of which are the regulations of the French, Prussian, Austrian and Swiss Governments, the Association of German Architects and Engineers, the German Concrete Association, and those proposed by a joint committee of the British Architectural and Building Associations and the Government Bureaus and the recommendations of the Special Committee on Concrete and Reinforced Concrete of the American Society of Civil Engineers, with such modifications as have been suggested by experience and the lessons taught by failures.

For that part of the specifications covering aggregates, preparation and placing of concrete and mortar, the recommendations of the Special Committee on Concrete and Reinforced Concrete of the American Society of Civil Engineers have been adopted as representing the best modern practice.

C. C. SCHNEIDER.

PHILADELPHIA, PA., May, 1910.

CONTENTS.

GENERAL SPECIFICATIONS FOR STRUCTURAL WORK OF BUILDINGS.

APPENDIX.

CONSISTING OF THE FOLLOWING TABLES:

GENERAL SPECIFICATIONS FOR STRUCTURAL WORK OF BUILDINGS.

DESIGN.

Loads.

1. The "dead" load in all structures shall consist of the weight of walls, floors, partitions, roofs and all other permanent construction and fixtures. **Dead Load.**

2. In calculating the "dead" loads, the weights of the different materials shall be assumed as given in Table 1.

3. The minimum weight of fire-proof floors to be assumed in designing the floor system shall be 75 lb. per sq. ft. For columns, the actual weight of floors shall be used.

4. For office buildings, 10 lb. per sq. ft. of floor area shall be added to the dead load of the floor for movable partitions.

5. The following table gives the "live" load on floors, to be assumed for different classes of buildings. These loads consist of: **Live Load on Floors.**

a.—A uniform load per square foot for floor area;

b.—A concentrated load which shall be applied to any point of the floor;

c.—A uniform load per linear foot for girders.

The maximum result is to be used in calculations.

The specified concentrated loads shall also apply to the floor construction between the beams for a length of 5 ft.

Table of Live Loads.

Classes of buildings.	Live Loads, in Pounds.		
	Distributed load.	Concentrated load.	Load per linear foot of girder.
Dwellings, hotels, apartment-houses, dormitories, hospitals	40	2 000	500
Office buildings, upper stories	50	5 000	1 000
Schoolrooms, theater galleries, churches	60	5 000	1 000
Ground floors of office buildings, corridors and stairs in public buildings	80	5 000	1 000
Assembly rooms, main floors of theaters, ballrooms, gymnasia, or any room likely to be used for drilling or dancing	floor 100, columns 50	5 000	1 000
Ordinary stores and light manufacturing, stables and carriage-houses	80	8 000	1 000
Sidewalks in front of buildings	300	10 000	1 000
Warehouses and factories	from 120 up	Special.	Special.
Charging floors for foundries	" 300 "	"	"
Power-houses, for uncovered floors	" 200 "	The actual weights of engines, boilers, stacks, etc., shall be used, but in no case less than 200 lb. per sq. ft.	

6. If heavy concentrations, like safes, armatures, or special machinery, are likely to occur on floors, provision should be made for them.

Crane Loads and Impact. 7. For structures carrying traveling machinery, such as cranes, conveyors, etc., 25% shall be added to the stresses resulting from such live load, to provide for the effects of impact and vibrations. (For crane loads, see Tables 12 and 13.)

Live Loads on Flat Roofs. 8. Flat roofs of office buildings, hotels, apartment-houses, etc., which can be loaded by crowds of people, shall be treated as floors, and the same distributed live loads shall be used as specified for hotels and dwelling-houses.

Wind Pressure. 9. The wind pressure shall be assumed acting in any direction horizontally:

First.—At 20 lb. per sq. ft. on the sides and ends of buildings and on the actually exposed surface, or the vertical projection of roofs;

Second.—At 30 lb. per sq. ft. on the total exposed surfaces of all parts composing the metal framework. The framework shall be considered an independent structure, without walls, partitions or floors.

Live Loads on Roofs. 10. Roofs shall be proportioned to carry in addition to their own weight the following live loads:

a.—A snow load, per horizontal square foot of roof, of 25 lb. for all slopes up to 20°; this load to be reduced 1 lb. for every degree of increase in the slope up to 45°, above which no snow load is considered.

b.—A wind load as specified in paragraph 9.

The possibility of a partial snow load has to be considered.

The above loads given for snow are the minimum values for localities where snow is likely to occur. In severe climates these snow loads should be increased in accordance with the actual conditions existing in those localities. In tropical climates the snow loads may be neglected.

Loads on Ordinary Roofs. 11. In climates corresponding to that of New York, ordinary roofs, up to 80 ft. span, shall be proportioned to carry the following minimum loads, per square foot of exposed surface, applied vertically, to provide for dead, wind and snow loads combined:

Gravel or Composition Roofing:	On boards, flat slope, 1 to 6, or less	50 lb.
	On boards, steep slope, more than 1 to 6	45 "
	On 3-in. flat tile or cinder concrete	60 "
Corrugated sheeting, on boards or purlins		40 "
Slate:	On boards or purlins	50 "
	On 3-in. flat tile or cinder concrete	65 "
Tile, on steel purlins		55 "
Glass		45 "

12. For roofs in climates where no snow is likely to occur, reduce the foregoing loads by 10 lb. per sq. ft., but no roof or any part thereof shall be designed for less than 40 lb. per sq. ft.

Live Loads on Columns. 13. For columns, the specified uniform live loads per square foot shall be used, with a minimum of 20,000 lb. per column.

Reduction of Live Load on Columns. 14. For columns carrying more than five floors, these live loads may be reduced as follows:

For columns supporting the roof and top floor, no reduction;

For columns supporting each succeeding floor, a reduction of 5% of the total live load may be made until 50% is reached, which reduced load shall be used for the columns supporting all remaining floors.

This reduction is not to apply to live load on columns of warehouses, and similar buildings which are likely to be fully loaded on all floors at the same time.

Loads on Foundations. 15. The live loads on foundations shall be assumed to be the same as for the footings of columns. The areas of the bases of the foundations shall be proportioned for the dead load only. That foundation which receives the largest ratio of live to dead load shall be selected and proportioned for the combined dead and live loads. The dead load on this foundation shall be divided by the area thus found, and this reduced pressure per square foot shall be the permissible working pressure to be used for the dead load of all foundations.

Unit Stresses and Proportion of Parts.

Substructure.

Foundations. 16. Pressure on foundations not to exceed, in tons per square foot:

Soft clay	1
Ordinary clay and dry sand mixed with clay	2
Dry sand and dry clay	3
Hard clay and firm, coarse sand	4
Firm, coarse sand and gravel	6

Masonry. 17. Working pressure in masonry not to exceed the following:

	Tons per sq. ft.	Lb. per sq. in.
Common brick, Portland-cement mortar	12	168
Hard-burned brick, Portland-cement mortar	15	210
Rubble masonry, Portland-cement mortar	10	140
Coursed rubble, Portland-cement mortar	12	168
First-class masonry, sandstone	20	280
" " " limestone or bluestone	25	350
" " " granite	30	420
Concrete for walls:		
Portland cement 1:2:5	20	280
" " 1:2:4	25	350

Pressure of Wall Plates. 18. The pressure of beams, girders, wall-plates, column bases, etc., on masonry shall not exceed the following, in pounds per square inch:

On brickwork with cement mortar	300
" rubble masonry with cement mortar	250
" Portland-cement concrete 1:2:4	600
" first-class sandstone (dimension stone)	400
" " " limestone	500
" " " granite	600

Bearing Power of Timber Piles. 19. The maximum load carried by any pile shall not exceed 40,000 lb., or 600 lb. per sq. in. of its average cross-section.

Piles driven in firm soil to rock may be loaded to the above limits. Piles driven through loose, wet soil to solid rock, or equivalent bearing, shall be figured as columns with a maximum unit stress of 600 lb. per sq. in., properly reduced.

Masonry Pillars and Walls Laid in Cement Mortar.

Pillars. 20. Pillars of brick or stone masonry, with concentric loading, may be built of a height not exceeding 12 times their diameter or their least lateral dimension; providing the unit pressure comes within the limits specified for the different classes of masonry.

21. The dimensions of pillars loaded eccentrically must be such that the center of pressure comes within the middle third of the base and every other horizontal section, and that the maximum unit pressure does not exceed the safe working pressure.

Walls. 22. The thickness of a wall depends upon the quality of the material used, the load it has to carry, and upon its unsupported height or length. The minimum thickness of a wall of brick or ashlar masonry shall be $\frac{1}{16}$ of its least unsupported distance, either vertically or horizontally; and that of walls of rubble masonry, $\frac{1}{8}$ of that distance.

23. The minimum thickness of brick enclosure walls shall be 12 in., and that of stone walls, 18 in. **Exterior Walls.**

24. The minimum thickness of curtain walls in the steel skeleton type of buildings shall be 12 in. **Curtain Walls.**

25. The unsupported height of a wall shall be taken as the height of one story, provided it is properly anchored to the floor construction of each story. The unsupported distance horizontally shall be taken as the distance between lateral walls which are properly bonded to it, or the distance between buttresses or steel columns.

26. In a wall carrying joists or beams, the load may be considered as distributed, if the distance between the beams is not more than twice the thickness of the wall. If a wall has to support concentrated loads, such as are produced by heavy roof trusses or floor girders, it must be reinforced by buttresses, which should be computed as pillars. **Bearing Walls.**

27. In the case of buildings several stories in height, the minimum thickness of the exterior walls supporting floors and roof may be approximately determined by the following empirical formula, which gives results agreeing with the provisions of most of the existing building laws.

28. The thickness of wall in inches $t = \frac{L}{4} + \frac{H_1 + H_2 + \ldots\ldots\ldots H_n}{6}$
where L = unsupported length in feet, which should not be assumed less than 24 ft., and H_1, H_2, H_3, etc., the heights of the stories in feet, commencing at the top.

29. The above rules apply to walls of brick and ashlar masonry for dwellings, hotels and office buildings.

30. The cellar wall shall generally be 4 in. thicker than the wall immediately above it, to a depth of 12 ft. below the grade line; and for every additional 10 ft., or part thereof, shall be increased 4 in. Cellar and foundation walls of masonry shall be 4 in. thicker than brick walls. **Cellar and Foundation Walls.**

31. If any horizontal section through any bearing wall shows more than 30% area of flues or openings, such wall shall be increased in thickness 1 in. for every 4%, or fraction thereof, by which the total areas of flues and openings exceed 30 per cent.

Non-bearing Walls.

32. The thickness of non-bearing walls may be 4 in. less than that of bearing walls, provided that no non-bearing wall is less than 12 in. thick.

Steel Superstructure.

Unit Stresses.

Permissible Stresses.

33. All parts of the structure shall be proportioned so that the sum of the dead and live loads, together with the impact, if any, shall not cause the stresses to exceed the following amounts in pounds per sq. in.:

Tension.

34. Tension, net section, rolled steel 16 000

Compression.

35. Direct compression, rolled steel and steel castings 16 000

Bending.

36. Bending, on extreme fibers of rolled shapes, built sections, girders and steel castings, net section 16 000

On extreme fibers of pins 24 000

Shear.

37. Shear, on rivets and pins 12 000

On bolts and field rivets 10 000

On plate-girder web (gross section) 10 000

Bearing.

38. Bearing pressure, on pins and rivets 24 000

On bolts and field rivets 20 000

Axial Compression.

39. Axial compression on gross section of columns ... $16\,000 - 70\,\frac{l}{r}$

with a maximum of 14 000

Where l = effective length* of member in inches;

r = corresponding radius of gyration of the section, in inches.

40. For bracing and the combined stresses due to wind and other loading, the permissible working stresses may be increased 25%, or to 20,000 lb. for direct compression or tension.

Provision for Eccentric Loading.

41. In proportioning columns, provision must be made for eccentric loading.

Expansion Rollers.

42. The pressure per linear inch on expansion rollers shall not exceed 600 d, where d = diameter of rollers, in inches.

Combined Stresses.

43. Members subject to the action of both axial and bending stresses shall be proportioned so that the greatest fiber stress will not exceed the allowed limits in that member.

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* The effective length "l", if L is the length of the member between centres of connections, shall be taken as follows:

$l = L$, if both ends are hinged or butting;
$l = \frac{1}{2} L$, if both ends are fixed;
$l = \frac{3}{4} L$, if one end is fixed, the other hinged;
$l = 2\ L$, if one end is fixed, the other free to move.

Alternate Stresses.

44. Members subject to alternate stresses of tension and compression shall be proportioned for the stress giving the largest section, but their connections shall be proportioned for the sum of the stresses.

Net Sections.

45. Net sections must be used in calculating tension members, and, in deducting the rivet holes, they must be taken $\frac{1}{8}$ in. larger than the nominal size of the rivets.

46. Pin-connected riveted tension members shall have a net section through the pin holes 25% in excess of the net section of the body of the member. The net section back of the pin hole shall be at least 0.75 of the net section through the pin hole.

Limiting Length of Members.

47. The effective length of main compression members shall not exceed 125 times their least radius of gyration, and those for wind and lateral bracing, 150 times their least radius of gyration.

48. The length of riveted tension members in horizontal or inclined positions shall not exceed 200 times their radius of gyration about the horizontal axis. The horizontal projection of the unsupported portion of the member is to be considered as the effective length.

Plate Girders.

49. Plate girders shall be proportioned on the assumption that one-eighth of the gross area of the web is available as flange area. The thickness of the web plate shall not be less than $\frac{1}{160}$ of the unsupported distance between flange angles.

Compression Flanges of Plate Girders.

50. The compression flange shall have at least the same sectional area as the tension flange; nor shall the strain per square inch on the gross area exceed $16\,000 - 200\,\frac{l}{b}$, if cover consists of flat plates, or $16\,000 - 150\,\frac{l}{b}$ if cover consists of a channel section, where l = unsupported distance, and b = width of flange in inches.

Web Stiffeners.

51. The web shall have stiffeners at the ends and inner edges of bearing plates, and at all points of concentrated loads, and also at intermediate points, when the thickness of the web is less than one-sixtieth of the unsupported distance between flange angles, generally not farther apart than the depth of the full web plate, with a maximum limit of 5 ft.

Rolled Beams.

52. I-beams, and channels used as beams or girders, shall be proportioned by their moments of inertia.

Limiting Depth of Beams and Girders.

53. The depth of rolled beams in floors shall be not less than one-twentieth of the span, and, if used as roof purlins, not less than one-thirtieth of the span.

54. In case of floors subject to shocks and vibrations, the depth of beams and girders shall be limited to one-fifteenth of the span. If shallower beams are used, the sectional area shall be increased until the maximum deflection is not greater than that of a beam having a depth of one-fifteenth of the span, but the depth of such beams and girders shall in no case be less than one-twentieth of the span.

Cast Iron.

Permissible Stresses.

55. Compression ....12 000 lb. per sq. in.
Tension .... 2 500 " " " "
Shear .... 1 500 " " " "

Timber.

Timber.

56. The timber parts of the structure shall be proportioned in accordance with the following stresses, given in pounds per square inch:

Kind of timber.	Transverse loading.	End bearing.	Columns under 10 diameters.	Bearing across fiber.	Shear along fiber.
White Oak....................	1 200	1 200	1 000	500	200
Long-Leaf Yellow Pine.......	1 500	1 500	1 000	350	100
White Pine and Spruce......	1 000	1 000	600	200	100
Hemlock......................	800	800	500	200	100

Timber Columns.

57. Columns may be used with a length not exceeding 45 times the least dimension. The unit stress for lengths of more than 10 times the least dimension shall be reduced by the following formula:

$$P = C - \frac{C}{100}\,\frac{l}{d}$$

Where C equals unit stresses, as given above for short columns;
l " length of column, in inches;
d " least side of column, in inches.

Planking.

58. For the thickness of floor and roof planking, see Table 6.

Details of Steel Construction.

Minimum Thickness of Material.

59. No steel of less than $\frac{1}{4}$ in. thickness shall be used, except for lining or filling vacant spaces.

Adjustable Members.

60. Adjustable members in any part of structures shall preferably be avoided.

Symmetrical Sections.

61. Sections shall preferably be made symmetrical.

62. The strength of connections shall be sufficient to develop the full strength of the member. **Connections.**

63. No connection, except lattice bars, shall have less than two rivets.

64. Floor beams shall generally be rolled-steel beams. **Floor Beams.**

65. For fire-proof floors, they shall generally be tied with tie-rods at intervals not exceeding eight times the depth of the beams. This spacing may be increased for floors which are not of the arch type of construction. Holes for tie-rods, where the construction of the floor permits, shall be spaced about 3 in. above the bottom of the beam.

66. When more than one rolled beam is used to form a girder, they shall be connected by bolts and separators at intervals of not more than 5 ft. All beams having a depth of 12 in. and more shall have at least two bolts to each separator. **Beam Girder.**

67. Wall ends of a sufficient number of joists and girders shall be anchored securely to impart rigidity to the structure. **Wall Ends of Beams and Girders.**

68. Wall-plates and column bases shall be constructed so that the load will be well distributed over the entire bearing. If they do not get the full bearing on the masonry, the deficiency shall be made good with Portland-cement mortar. **Wall-Plates and Column Bases.**

69. The floor girders may be rolled beams or plate girders; they shall preferably be riveted or bolted to columns by means of connection angles. Shelf angles or other supports may be provided for convenience during erection. **Floor Girders.**

70. The flange plates of all girders shall be limited in width, so as not to extend beyond the outer line of rivets connecting them to the angles more than 6 in., or more than eight times the thickness of the thinnest plate. **Flange Plates.**

71. Web stiffeners shall be in pairs, and shall have a close bearing against the flange angles. Those over the end bearing, or forming the connection between girder and column, shall be on fillers. Intermediate stiffeners may be on fillers or crimped over the flange angles. The rivet pitch in stiffeners shall not be more than 5 in. **Web Stiffeners.**

72. Web plates of girders must be spliced at all points by a plate on each side of the web, capable of transmitting the full stress through splice rivets. **Web Splices.**

73. Columns shall be designed so as to provide for effective connections of floor beams, girders or brackets. **Columns.**

They shall preferably be continuous over several stories.

Column Splices.

74. The splices shall be strong enough to resist the bending stress and make the columns practically continuous for their whole length.

Trusses.

75. Trusses shall preferably be riveted structures. Heavy trusses, of long span, where the riveted field connections would become unwieldy, or for other good reasons, may be designed as pin-connected structures.

Intersecting Members.

76. Main members of trusses shall be designed so that the neutral axes of intersecting members shall meet in a common point.

Roof Trusses.

77. Roof trusses shall be braced in pairs in the plane of the chords. Purlins shall be made of rolled shapes, plate girders or lattice girders.

Eye-Bars.

78. The eye-bars in pin-connected trusses composing a member shall be as nearly parallel to the axis of the truss as possible.

Spacing of Rivets.

79. The minimum distance between centers of rivet holes shall be three diameters of the rivet; but the distance shall preferably be not less than 3 in. for $\frac{7}{8}$-in. rivets, $2\frac{1}{2}$ in. for $\frac{3}{4}$-in. rivets, and $1\frac{3}{4}$ in. for $\frac{1}{2}$-in. rivets. The maximum pitch in the line of the stress for members composed of plates and shapes shall be 6 in. for $\frac{7}{8}$-in. rivets, 5 in. for $\frac{3}{4}$-in. rivets, $4\frac{1}{2}$ in. for $\frac{5}{8}$-in. rivets and 4 in. for $\frac{1}{2}$-in. rivets.

80. For angles with two gauge lines, with rivets staggered, the maximum in each line shall be twice as great as given in Paragraph 79; and, where two or more plates are used in contact, rivets not more than 12 in. apart in either direction shall be used to hold the plates together.

81. The pitch of the rivet, in the direction of the stress, shall not exceed 6 in., nor 16 times the thinnest outside plate connected, and not more than 50 times that thickness at right angles to the stress.

Edge Distance.

82. The minimum distance from the center of any rivet hole to a sheared edge shall be $1\frac{1}{2}$ in. for $\frac{7}{8}$-in. rivets, $1\frac{1}{4}$ in. for $\frac{3}{4}$-in. rivets, $1\frac{1}{8}$ in. for $\frac{5}{8}$-in. rivets, and 1 in. for $\frac{1}{2}$-in. rivets; and to a rolled edge, $1\frac{1}{4}$, $1\frac{1}{8}$, 1 and $\frac{7}{8}$ in., respectively.

83. The maximum distance from any edge shall be eight times the thickness of the plate.

Maximum Diameter.

84. The diameter of the rivets in any angle carrying calculated stresses shall not exceed one-quarter of the width of the leg in which they are driven. In minor parts, rivets may be $\frac{1}{8}$ in. greater in diameter.

85. The pitch of rivets at the ends of built compression members shall not exceed four diameters of the rivets for a length equal to one and one-half times the maximum width of the member. **Pitch at Ends.**

86. The open sides of compression members shall be provided with lattice, having tie-plates at each end and at intermediate points where the lattice is interrupted. The tie-plates shall be as near the ends as practicable. In main members, carrying calculated stresses, the end tie-plates shall have a length not less than the distance between the lines of rivets connecting them to the flanges, and intermediate ones not less than half this distance. **Tie-Plates.**

Their thickness shall be not less than one-fiftieth of the same distance.

87. The latticing of compression members shall be proportioned to resist the shearing stresses corresponding to the allowance for flexure provided in the column formula in Paragraph 39 by the term $70 \frac{l}{r}$. The minimum thickness of lattice bars shall be one-fortieth for single lattice and one-sixtieth for double lattice, of the distance between end rivets; their minimum width shall be as follows: **Lattice.**

For 15-in. channels, or built sections with 3½ and 4-in. angles..........................	2½ in. (⅞-in. rivets);
For 12, 10 and 9-in. channels, or built sections with 3-in. angles..........................	2¼ in. (¾-in rivets);
For 8 and 7-in. channels, or built sections with 2½-in. angles..........................	2 in. (⅝-in. rivets);
For 6 and 5-in. channels, or built sections with 2-in. angles..........................	1¾ in. (½-in. rivets).

88. Lattice bars with two rivets shall generally be used in flanges more than 5 in. wide.

89. The inclination of lattice bars with the axis of the member, shall generally be not less than 45°, and when the distance between the rivet lines in the flange is more than 15 in., if a single rivet bar is used, the lattice shall be double and riveted at the intersection. **Angle of Lattice.**

90. The pitch of lattice connections, along the flange, divided by the least radius of gyration of the member between connections, shall be less than the corresponding ratio of the member as a whole. **Spacing of Lattice.**

Faced Joints. 91. Abutting joints in compression members when faced for bearing shall be spliced sufficiently to hold the connecting members accurately in place.

92. All other joints in riveted work, whether in tension or compression, shall be fully spliced.

Pin Plates. 93. Pin holes shall be reinforced by plates where necessary; and at least one plate shall be as wide as the flange will allow; where angles are used, this plate shall be on the same side as the angles. The plates shall contain sufficient rivets to distribute their portion of the pin pressure to the full cross-section of the member.

Pins. 94. Pins shall be long enough to insure a full bearing of all parts connected upon the turned-down body of the pin.

95. Members packed on pins shall be held against lateral movement.

Bolts. 96. Where members are connected by bolts, the body of these bolts shall be long enough to extend through the metal. A washer at least $\frac{3}{16}$ in. thick shall be used under the nut.

Fillers. 97. Fillers between parts carrying stress shall have a sufficient number of independent rivets to transmit the stress to the member to which the filler is attached.

Temperature. 98. Provision shall be made for expansion and contraction, corresponding to a variation of temperature of 150° Fahr. where necessary.

Rollers. 99. Expansion rollers shall be not less than 4 in. in diameter.

Stone Bolts. 100. Stone bolts shall extend not less than 4 in. into granite pedestals and 8 in. into other material.

Anchorage. 101. Columns which are strained in tension at their base shall be anchored to the foundations.

102. Anchor bolts shall be long enough to engage a mass of masonry, the weight of which shall be one and one-half times the tension in the anchor.

Bracing. 103. Lateral, longitudinal and transverse bracing in all structures shall preferably be composed of rigid members.

MATERIAL AND WORKMANSHIP.

Material.

Steel. 104. All parts of the metallic structure shall be of rolled steel, except column bases, bearing plates or minor details, which may be of cast iron or cast steel.

105. Steel may be made by the open-hearth or by the Bessemer process. Process of Manufacture.

106. The chemical and physical properties shall conform to the following limits: Requirements.

Chemical and physical properties.	Structural steel.	Rivet steel.	Steel castings.
Phosphorus, maximum..................	0.04%	0.04%	0.05%
Sulphur, maximum.....................	0.05%	0.04%	0.05%
Ultimate tensile strength; pounds per square inch..........................	Desired 60 000	Desired 50 000	Not less than 65 000
Elongation: minimum percentage in 8 in..................................	1 500 000* / Ultimate tensile strength.	1 500 000 / Ultimate tensile strength.	....
Elongation: minimum percentage in 2 in.	22	....	18
Character of fracture......	Silky.	Silky.	Silky or fine granular.
Cold bends without fracture.............	180° flat.†	180° flat.§	90°

* See Paragraph 117. † See Paragraphs 118, 119 and 120. § See Paragraph 121.

107. In order that the ultimate strength of full-sized annealed eye-bars may meet the requirements of Paragraph 170, the ultimate strength in test specimens may be determined by the manufacturers; all other tests than those for ultimate strength shall conform to the above requirements.

108. The yield point, as indicated by the drop of beam, shall be recorded in the test reports.

109. Tensile tests of steel showing an ultimate strength within 5,000 lb. of that desired will be considered satisfactory. Allowable Variations.

110. Chemical determinations of the percentages of carbon, phosphorus, sulphur and manganese shall be made by the manufacturer from a test ingot taken at the time of the pouring of each melt of steel, and a correct copy of such analysis shall be furnished to the engineer or his inspector. Chemical Analyses.

111. Specimens for tensile and bending tests for plates, shapes and bars shall be made by cutting coupons from the finished product, which shall have both faces rolled and both edges milled to the form shown by Fig. 1; or with both edges parallel; or they may be turned to a diameter of ¾ in. for a length of at least 9 in., with enlarged ends. Form of Specimens for Plates, Shapes and Bars.

112. Rivet rods shall be tested as rolled. Rivets.

113. Specimens shall be cut from the finished rolled or forged bar, in such manner that the center of the specimen shall be 1 in. from the surface of the bar. The specimen for the tensile test shall be Pins and Rollers.

turned to the form shown by Fig. 2. The specimen for the bending test shall be 1 in. by ½ in. in section.

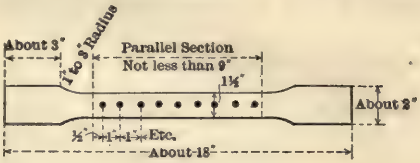

FIG. 1.

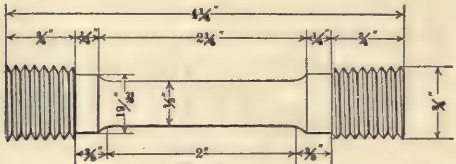

FIG. 2.

Steel Castings. 114. The number of tests will depend on the character and importance of the castings. Specimens shall be cut cold from coupons moulded and cast on some portion of one or more castings from each melt, or from the sink-heads, if the heads are of sufficient size. The coupon or sink-head, so used, shall be annealed with the casting before it is cut off. Test specimens shall be of the form prescribed for pins and rollers.

Specimens of Rolled Steel. 115. Rolled steel shall be tested in the condition in which it comes from the rolls.

Number of Tests. 116. At least one tensile and one bending test shall be made from each melt of steel as rolled. In case steel differing ⅜ in. and more in thickness is rolled from one melt, a test shall be made from the thickest and thinnest material rolled.

Modifications in Elongation. 117. For material more than ¾ in. in thickness, a deduction of 1% will be allowed from the specified elongation for each ⅛ in. in thickness above ¾ in.

Bending Tests. 118. Bending tests may be made by pressure or by blows. Plates, shapes and bars less than 1 in. thick shall bend as called for in Paragraph 106.

Thick Material. 119. Full-sized material for eye-bars and other steel 1 in. or more in thickness, tested or rolled, shall bend cold 180° around a pin, the diameter of which is equal to twice the thickness of the bar, without fracture on the outside of the bend.

Bending Angles. 120. Angles ¾ in. and less in thickness shall open flat, and angles ½ in. and less in thickness shall bend shut, cold, under blows of a hammer, without sign of fracture. This test will be made only when required by the inspector.

121. Rivet steel, when nicked and bent around a bar of the same diameter as the rivet rod, shall give a gradual break and a fine, silky, uniform fracture. **Nicked Bends.**

122. Finished material shall be free from injurious seams, flaws, cracks, defective edges, or other defects, and shall have a smooth, uniform, workmanlike finish. Plates 36 in. and less in width shall have rolled edges. **Finish.**

123. Every finished piece of steel shall have the melt number and the name of the manufacturer stamped or rolled upon it. Steel for pins and rollers shall be stamped on the end. Rivet and lattice steel and other small parts may be bundled with the above marks on an attached tag. **Stamping.**

124. Material which, subsequent to the foregoing tests at the mills, and its acceptance there, develops weak spots, brittleness, cracks or other imperfections, or is found to have injurious defects, will be rejected at the shop, and shall be replaced by the manufacturer at his own cost. **Defective Material.**

125. A variation in cross-section or weight in the finished members of more than $2\frac{1}{2}\%$ from that specified will be sufficient cause for rejection. **Allowable Variation in Weight.**

126. Iron castings shall be made of tough, gray iron, free from injurious cold-shuts or blow-holes, true to pattern and of workmanlike finish. Test pieces $1\frac{1}{4}$ in. round shall be capable of sustaining on a clear span of 12 in. a central load of at least 2 900 lb., and deflect at least $\frac{1}{10}$ in. before rupture. **Cast Iron.**

Workmanship.

127. All parts forming a structure shall be built in accordance with approved drawings. The workmanship and finish shall be equal to the best practice in modern bridge works. **General.**

128. Material shall be thoroughly straightened in the shop, by methods which will not injure it, before being laid off or worked in any way. **Straightening Material.**

129. Shearing shall be done neatly and accurately, and all portions of the work exposed to view shall be neatly finished. **Finish.**

130. The size of rivets called for on the plans shall be understood to mean the actual size of the cold rivet before heating. **Rivets.**

Rivet Holes. 131. The diameter of the punch for material not more than $\frac{3}{4}$ in. thick shall be not more than $\frac{1}{16}$ in., nor that of the die more than $\frac{1}{8}$ in. larger than the diameter of the rivet. Material more than $\frac{3}{4}$ in. thick, excepting in minor details, shall be sub-punched and reamed or drilled from the solid.

Punching. 132. Punching shall be done accurately. Slight inaccuracy in the matching of holes may be corrected with reamers. Drifting to enlarge unfair holes will not be allowed. Poor matching of holes will be cause for rejection, at the option of the inspector.

Assembling. 133. Riveted members shall have all parts well pinned up and firmly drawn together with bolts before riveting is commenced. Contact surfaces shall be painted. (See Paragraph 157.)

Lattice Bars. 134. Lattice bars shall have neatly rounded ends, unless otherwise called for.

Web Stiffeners. 135. Stiffeners shall fit neatly between the flanges of girders. Where tight fits are called for, the ends of the stiffeners shall be faced and shall be brought to a true contact bearing with the flange angles.

Splice Plates and Fillers. 136. Web splice plates and fillers under stiffeners shall be cut to fit within $\frac{1}{8}$ in. of flange angles.

Connection Angles. 137. Connection angles for floor girders shall be flush with each other and correct as to position and length of girder.

Riveting. 138. Rivets shall be driven by pressure tools wherever possible. Pneumatic hammers shall be used in preference to hand driving.

Heating of Rivets. 139. Rivets shall be heated to a light cherry-red heat in a gas or oil furnace. The furnace must be so constructed that it can be adjusted to the proper temperature.

Rivets. 140. Rivets shall look neat and finished, with heads of approved shape, full, and of equal size. They shall be central on the shank and shall grip the assembled pieces firmly. Recupping and caulking will not be allowed. Loose, burned, or otherwise defective rivets shall be cut out and replaced. In cutting out rivets, great care shall be taken not to injure the adjoining metal. If necessary, they shall be drilled out.

Field Bolts. 141. Wherever bolts are used in place of rivets which transmit shear, such bolts must have a driving fit. A washer not less than $\frac{1}{4}$ in. thick shall be used under the nut.

Members to be Straight. 142. The several pieces forming one built member shall be straight and shall fit closely together, and finished members shall be free from twists, bends or open joints.

143. Abutting joints shall be cut or dressed true and straight and fitted closely together, especially where open to view. In compression joints depending on contact bearing, the surfaces shall be truly faced, so as to have even bearings after they are riveted up complete and when perfectly aligned. Finish of Joints.

144. Eye-bars shall be straight and true to size, and shall be free from twists, folds in the neck or head, or any other defect. Heads shall be made by upsetting, rolling or forging. Welding will not be allowed. The form of the heads will be determined by the dies in use at the works where the eye-bars are made, if satisfactory to the engineer, but the manufacturer shall guarantee the bars to break in the body when tested to rupture. The thickness of the head and neck shall not vary more than $\frac{1}{16}$ in. from that specified. Eye-Bars.

145. Before boring, each eye-bar shall be perfectly annealed and carefully straightened. Pin holes shall be in the center line of bars and in the center of heads. Bars of the same length shall be bored so accurately that, when placed together, pins $\frac{1}{32}$ in. smaller in diameter than the pin holes can be passed through the holes at both ends of the bars at the same time. Boring Eye-Bars.

146. Pin holes shall be bored true to gauges, smooth and straight; at right angles to the axis of the member, and parallel to each other, unless otherwise called for. Wherever possible, the boring shall be done after the member is riveted up. Pin Holes.

147. The distance from center to center of pin holes shall be correct within $\frac{1}{32}$ in., and the diameter of the hole not more than $\frac{1}{50}$ in. larger than that of the pin, for pins up to 5 in. diameter, and $\frac{1}{32}$ in. for larger pins. Variation in Pin Holes.

148. Pins and rollers shall be turned accurately to gauges, and shall be straight, smooth and entirely free from flaws. Pins and Rollers.

149. At least one pilot and driving nut shall be furnished for each size of pin for each structure. Pilot Nuts.

150. Screw threads shall make tight fits in the nuts, and shall be United States standard, except for diameters greater than $1\frac{3}{8}$ in., when they shall be made with six threads per inch. Screw Threads.

151. Steel, except in minor details, which has been partially heated shall be properly annealed. Annealing.

152. All steel castings shall be annealed. Steel Castings.

153. Welds in steel will not be allowed. Welds.

Bed-Plates. 154. Expansion bed-plates shall be planed true and smooth. Cast wall-plates shall be planed at top and bottom. The cut of the planing tool shall correspond with the direction of expansion.

Shipping Details. 155. Pins, nuts, bolts, rivets and other small details shall be boxed or crated.

Painting.

156. Steelwork, before leaving the shop, shall be thoroughly cleaned and given one good coating of pure linseed oil, or such paint as may be called for, well worked into all joints and open spaces.

157. In riveted work, the surfaces coming in contact shall be painted before being riveted together.

Shop Painting. 158. Pieces and parts which are not accessible for painting after erection shall have two coats of paint before leaving the shop.

159. Steelwork to be entirely embedded in concrete shall not be painted.

160. Painting shall be done only when the surface of the metal is perfectly dry. It shall not be done in wet or freezing weather, unless protected under cover.

161. Machine-finished surfaces shall be coated with white lead and tallow before shipment, or before being put out into the open air.

Field Painting. 162. After the structure is erected, the metal-work shall be painted thoroughly and evenly with an additional coat of paint, mixed with pure linseed oil, of such quality and color as may be selected. Succeeding coats of paint shall vary somewhat in color, in order that there may be no confusion as to the surfaces which have been painted.

Inspection and Testing.

Facilities for Inspection. 163. The manufacturer shall furnish all facilities for inspecting and testing the weight, quality of material and workmanship. He shall furnish a suitable testing machine for testing the specimens, as well as prepare the pieces for the machine, free of cost.

Access to Shop. 164. When an inspector is furnished by the purchaser, he shall have full access at all times to all parts of the works where material under his inspection is manufactured.

Mill Orders. 165. The purchaser shall be furnished with complete copies of mill orders, and no material shall be rolled and no work done before he has been notified as to where the orders have been placed, so that he may arrange for the inspection.

166. The purchaser shall also be furnished with complete shop plans, and must be notified well in advance of the start of the work in the shop, in order that he may have an inspector on hand to inspect the material and workmanship. Shop Plans.

167. Complete copies of shipping invoices shall be furnished to the purchaser with each shipment. Shipping Invoices.

168. If the inspector, through an oversight or otherwise, has accepted material or work which is defective or contrary to the specifications, this material, no matter in what stage of completion, may be rejected by the purchaser. Accepting Material or Work.

Full-Sized Tests.

169. Full-sized tests on eye-bars and similar members, to prove the workmanship, shall be made at the manufacturer's expense, and shall be paid for by the purchaser at contract price, if the tests are satisfactory. If the tests are not satisfactory, the members represented by them will be rejected.

170. In eye-bar tests, the minimum ultimate strength shall be 55 000 lb. per sq. in. The elongation in 10 ft., including fracture, shall be not less than 15%. Bars shall break in the body and the fracture shall be silky or fine granular, and the elastic limit as indicated by the drop of the mercury shall be recorded. Should a bar break in the head and develop the specified elongation, ultimate strength and character of fracture, it shall not be cause for rejection, provided not more than one-third of the total number of bars break in the head.

CONCRETE AND REINFORCED CONCRETE.

Concrete, plain and reinforced, may now be considered one of the recognized materials of construction. It has proved to be satisfactory material, when properly used, for those purposes for which its qualities make it particularly suitable.

Proper Use.

Concrete is a material of very low tensile strength and capable of sustaining but very small tensile deformations without rupture; its value as a structural material depends chiefly upon its durability, its fire-resisting qualities, its strength in compression and its relatively low cost. Its strength generally increases with age.

Plain, or massive, concrete is well adapted for the construction of massive structural parts, which have to resist compression only, and as a substitute for stone or brick masonry in foundations, walls, piers, arches, culverts, docks, dams, reservoirs, sewers, tunnel linings, etc.

For such purposes concrete has stood the test of time, and may be used without reinforcement in blocks, or as a monolith. It has these advantages over stone masonry, that material for the aggregate can be found in almost any locality, and the concrete can easily be put in place, under proper supervision, without skilled workmen. Concrete in monolithic form is better adapted to receive reinforcement than stone masonry.

In substructures and foundations, the bases can be more conveniently and effectively enlarged by reinforcing. For certain kinds of masonry construction for which concrete is now extensively substituted, such as dams, retaining walls, etc., engineers have been able, by the use of proper reinforcement, to depart from the usual forms of construction and adopt new ones.

Owing to its fire-resisting qualities, reinforced concrete is a suitable material for fire-proof construction for floor and roof slabs, curtain walls, partitions, etc.

Improper Use.

Failures of reinforced concrete structures are usually due to any one or a combination of the following causes: Defective design, poor material and faulty execution.

The defects in a design may be many and various. The computations and assumptions on which they are based may be faulty and contrary to the established principles of statics; the unit stresses used may be excessive, or the details of the design defective.

As the properties of concrete and reinforced concrete are not yet as well understood and clearly defined as those of steel, owing to the lack of conclusive tests and experience, and as there is no generally accepted theory in existence at the present time for computing the interior forces in reinforced concrete structures, the data which are now available should be used with caution, so that if there be an error, it will be on the side of safety.

The design of a structure built of reinforced concrete should, therefore, receive at least the same careful consideration as one of steel, and only engineers with sufficient experience and good judgment should be intrusted with such work.

The computations should include all the minor details, which are sometimes of the utmost importance. The design should show clearly the size and position of the reinforcement, and should provide for proper connections between the component parts so that they cannot be displaced. The best results are obtained when the reinforcement of any member is a unit, so that the reinforcement can be put in position without depending on the laborers to put each bar in its proper place. As the connections between the members are generally the weakest points, the design should provide for proper attachments between the reinforcements of connecting members and should be accompanied by computations to prove their strength.

The use of unwarranted high unit stresses, approaching the danger line, is one of the common defects in the design of reinforced concrete structures.

Articulated concrete structures designed in imitation of steel trusses may be mentioned as illustrating the improper use of reinforced concrete. Long concrete columns, reinforced with longitudinal round or square bars intended to take compression, but which cannot resist buckling, may also be mentioned in this connection.

Poor material is sometimes used for the concrete, as well as for the reinforcement. Poor concrete is not always used intentionally, but is often allowed to go into the structure owing to the lack of experience of the contractor and his superintendents, or to the absence of proper supervision.

A poor quality of steel for reinforcement is sometimes called for in the specifications for the purpose of reducing the cost. For steel structures, a high grade of material is used, while the steel used for reinforcing concrete is sometimes made of old rails or other unsuitable, brittle material, which is not fit to be used in any permanent structure.

Faulty execution and careless workmanship may generally be attributed to unintelligent, insufficient supervision.

The remarks referring to the improper uses of reinforced concrete apply more particularly to building construction, where rational design, good material, good workmanship and adequate supervision are the exception rather than the rule.

While other structures upon the safety of which human lives depend are generally designed by engineers employed by the owner, and the contracts let on the engineer's design and specifications, in accordance with legitimate practice, reinforced concrete structures are as a rule designed by contractors or engineers commercially interested, and the contract let for a lump sum, without the advice of a competent engineer, and regardless of the merits of the design.

The construction of buildings in large cities is regulated by municipal authorities. For reinforced concrete work, however, the limited supervision which municipal inspectors are able to give is not sufficient. Other means for more adequate supervision and inspection should, therefore, be provided.

Responsibility and Supervision.

If any failure occurs in an important engineering structure, the engineer is generally held responsible for the same. In recent failures of reinforced concrete buildings, coroners' juries either put the responsibility on unknown causes, or on some ignorant, innocent subordinate, who had to act as scapegoat for his employer.

Disasters have proved that the execution of the work should not be separated from the designing of the structure. Intelligent, rational supervision and execution of the work can be expected only when both functions are combined. The engineer who prepares the design and specifications should also have the supervision of the execution of the work, and may then he held responsible for its entire construction, unless it can be proven that the contractor has done work contrary to design, specifications and orders of the engineer, which the engineer and his inspectors were unable to prevent. In this case the contractor should be held responsible.

For the purpose of fixing the responsibility and providing for adequate supervision during construction, the Special Committee on Concrete and Reinforced Concrete of the American Society of Civil Engineers recommends the following rules:

a. Before work is commenced, complete plans shall be prepared, accompanied by specifications, static computations and descriptions showing the general arrangement and all details. The static computations shall give the loads assumed separately, such as dead and live loads, wind and impact, if any, and the resulting stresses.

b. The specifications shall state the qualities of the materials to be used for making the concrete, and the manner in which they are to be proportioned.

c. The strength which the concrete is expected to attain after a definite period shall be stated in the specifications.

d. The drawings and specifications shall be signed by the engineer and the contractor.

e. The approval of plans and specifications by other authorities shall not relieve the engineer nor the contractor of responsibility.

SPECIFICATIONS FOR PLAIN AND REINFORCED CONCRETE CONSTRUCTION.

The following tentative specifications apply to all structures, or parts thereof, built of plain or reinforced concrete:

Design.

1. In the design of massive concrete or plain concrete, no account should be taken of the tensile strength of the material, and sections should usually be so proportioned as to avoid tensile stresses. This will generally be accomplished, in the case of rectangular shapes, if the line of pressure is kept within the middle third of the section, but in very large structures, such as high masonry dams, a more exact analysis may be required. Structures of massive concrete are able to resist unbalanced lateral forces by reason of their weight, hence the element of weight rather than strength often determines the design. A relatively cheap and weak concrete will therefore often be suitable for massive concrete structures. Owing to its low extensibility, the contraction due to hardening and to temperature changes requires special consideration, and, except in the case of very massive walls, such as dams, it is desirable to provide joints at intervals to localize the effect of such contraction. The spacing of such joints will depend upon the form and dimensions of the structure and its degree of exposure. **Massive Concrete.**

2. Massive concrete may be used for piers and short columns in which the ratio of length to least width is relatively small. Under ordinary conditions this ratio should not exceed six, but, where the central application of the load is assured, a somewhat higher value may safely be used.

3. Massive concrete is also a suitable material for arches of moderate span where the conditions as to foundations are favorable.

4. By the use of metal reinforcement to resist the principal tensile stresses, concrete becomes available for general use in a great variety of structures and structural forms. This combination of concrete and steel may be used to advantage in the beam, where both compression and tension exist; and the column, where the main stresses are compressive, but where cross-bending may exist. The theory of design will therefore relate mainly to the analysis of beams and columns. **Reinforced Concrete.**

General Assumptions for Static Computations.

External Forces.

Loads.

5. Buildings of reinforced concrete are to be designed for the same vertical loads and wind pressure as specified on pages 7-9, the weight of reinforced concrete to be assumed at 150 lb. per cu. ft.

Reactions, Moments, Shear.

6. For the computations of the end reactions, moments and shear, the established rules of statics and of elasticity shall be followed.

7. In order to obtain the maximum values, the most unfavorable positions and distributions of the live load must be considered.

8. Possible effects of impact may be considered by adding the usual percentage to the live load.

9. The span length for computations is to be taken as follows:

a. For beams, the distance between centers of supports; but shall not be taken to exceed the clear span plus the depth of the beam.

b. For freely supported floor slabs, the clear span plus the thickness of the slab in the center.

c. For continuous slabs, the distance center to center of beams.

Continuous beams and slabs.

10. For continuous beams and slabs, the bending moment at center and at support shall be taken as $\frac{4}{5}$ of the moment of a freely supported beam of the same span.

Slabs reinforced in both directions.

11. For square floor slabs reinforced in both directions and supported on all sides, the bending moment may be taken as $\frac{2}{5}$ of that of a freely supported beam of the same length.

Columns.

12. In computing the strength of columns, the possibility of eccentric loading must be considered.

T-beams.

13. In the design of T-beams acting as continuous beams, due consideration should be given to the compressive stresses at the supports. For beams of T-sections, the width of the floor slab to be considered as part of the beam shall not be more than 8 times the thickness of the slab, or $\frac{1}{3}$ of the span length of the beam.

Internal Stresses.

14. The internal stresses in reinforced concrete structures shall be determined the same as in the case of homogenous material on the following assumptions:

15. (*a*) The stress in any fiber is directly proportionate to the distance of that fiber from the neutral axis.

16. (*b*) The modulus of elasticity of the concrete remains constant within the limits of the working stresses fixed in these specifications. In compression, the two materials are, therefore, strained in proportion to their moduli of elasticity.

17. (*c*) The bond between the concrete and steel is sufficient to make the two materials act together as a homogeneous solid.

Moduli of Elasticity.

18. The ratio of the modulus of elasticity of steel to the modulus of elasticity of stone concrete may be taken at 15, and of cinder concrete at 30.

19. The tensile strength of the concrete shall be neglected.

20. When the shearing stresses developed in any part of the constuction exceed the safe working strength of concrete as specified, a sufficient amount of reinforcement shall be introduced in such manner that the deficiency in the resistance to shearing is overcome.

21. When the safe limit of bond between the concrete and the steel is exceeded, some provision must be made for transmitting the strength of the steel to the concrete.

Reinforced Columns.

22. For columns reinforced with shapes that can resist buckling, the computations may be made in the same manner as for homogeneous material, if, in the areas and moments of resistence, the section of steel reinforcement is added to that of the concrete with 15 times its value.

23. In columns with concentric loading, buckling need not be considered if the ratio of the effective length to the effective diameter does not exceed 12. The effective diameter to correspond to the assumed theoretical area.

24. If tensile stresses produced by eccentric loads or bending moments occur in a column, the steel reinforcement on the tension side must be able to resist the same.

Working Stresses.

25. The following working stresses are recommended for static loads:

Steel.

26. For the steel reinforcement, the unit stresses shall not exceed those specified for other structural steel work. (Paragraph 33, page 12.)

Concrete.

27. The following working stresses for concrete are based on the

compressive strength of the concrete, developed after 28 days, when tested in cylinders 8 in. in diameter and 16 in. long:

*Bearing	30%	of the compressive strength.
Compression in extreme fiber	25%	" " " "
Axial compression in columns	20%	" " " "
Shear	3%	" " " "
Bond, rolled bars	3%	" " " "
" drawn wire	2%	" " " "

Stone Concrete.

28. For stone concrete composed of one part Portland cement and 6 parts aggregate, capable of developing an average compressive strength of 2 000 lb. per sq. in., at 28 days, the working stresses shall not exceed the following:

Bearing	600	lb. per sq. in.
Compression in extreme fiber	500	" " "
Axial compression in columns	400	" " "
Shear	60	" " "
Bond, rolled bars	60	" " "
" drawn wire	40	" " "

Cinder Concrete.

29. For cinder concrete capable of developing an average compressive strength of 750 lb. per sq. in., at 28 days, the working stresses shall not exceed the following:

Bearing	225	lb. per sq. in.
Compression in extreme fiber	185	" " "
Shear	25	" " "
Bond	30	" " "

Working Stresses on Reinforced Columns.

Reinforced Columns.

30. For axial compression on concrete in columns reinforced against buckling, the same working stresses as those recommended for bearing may be used. If in columns reinforced against buckling the reinforcement is tied together, so that the concrete may be considered as restrained similarly to concrete enclosed in a steel tube, the working strain on the concrete may be increased to 35% of its compressive strength, or approximately 700 lb. per sq. in. for 2 000 lb. concrete.

* Compression applied to a surface of concrete larger than the loaded area, such as the pressure on bed-plates.

Details of Construction.

31. The specifications for the design of structural steel work shall also apply to the steel reinforcement of concrete construction. **Steel Work.**

32. Plain concrete columns may be used, if the ratio of length to the least side or diameter does not exceed 12, without any reduction in the working stress specified for axial compression. **Plain Concrete Columns.**

33. The reinforcement of columns shall consist of shapes which can resist compression. These shapes shall be rigidly connected by lattice bars or tie-plates at proper intervals, so as to form a skeleton column. Only such columns shall be considered as reinforced. **Column Reinforcement.**

34. The reinforcement should be provided with proper connections between the bars to hold them in the right place and at the correct distance from the nearest face of the concrete, so as to prevent dislodgment during the depositing and compacting of the concrete. **Beam Reinforcement.**

35. If the reinforcement consists of round or square bars, their lateral spacing should not be less than 1½ diameters, center to center; nor should the distance from the side of the beam to the center of the nearest bar be less than 2 diameters.

36. When the beam or slab is continuous over its support, reinforcement should be provided at points of negative moment. **Continuous Beams and Slabs.**

37. In connections between members, such as between columns and girders, and girders and beams, the reinforcements of the connecting members shall be firmly attached to each other. **Connections Between Members.**

38. The concrete outside of the reinforcement is not to be considered as carrying any load.

39. Plain concrete walls, if made of concrete which will develop an average compressive strength of at least 1 500 lb. per sq. in. after 28 days, may be of the same thickness as brick walls laid in cement mortar. If properly reinforced in both directions, the thickness may be reduced to two-thirds of that of brick walls. Spandrel and curtain walls of steel skeleton construction shall have a minimum thickness of 8 in. if reinforced with not less than ¾ lb. of steel per sq. ft. of wall. Partitions, if constructed of reinforced concrete, shall have a minimum thickness of 3 in., and shall be reinforced with not less than ¼-in. rods on 12-in. centers, running both vertically and horizontally. The filling of panels of the skeleton frames of sheds or mill buildings shall not be less than 4 in. **Walls.**

Fireproofing. 40. In plain concrete columns, the concrete to a depth of 1½ in. may be considered as protective covering, and should not be included in the effective section. Under ordinary conditions, the concrete covering over the metal reinforcement in office buildings, hotels and similar structures should be at least 2 in. for girders and columns, 1½ in. for beams, and 1 in. for floor slabs. In stores, warehouses or other buildings where combustible materials are likely to be stored, the thickness of the protection should be increased to 3 or 4 in.

MATERIALS AND WORKMANSHIP.

Stone Concrete. 41. Stone or gravel concrete shall be used in the construction of girders and columns, or any other parts which carry loads or constitute integral parts of the structure.

Cinder Concrete. 42. Cinder concrete may be used for fireproofing, for floor slabs and for parts which do not carry any loads, such as curtain walls, spandrel walls, parapet walls, partitions and filling of panels of steel skeletons of sheds or mill building.

Portland Cement. 43. Only Portland cement conforming to the standard specifications of the American Society for Testing Materials shall be used in reinforced concrete work.

AGGREGATES.

44. Extreme care should be exercised in selecting the aggregates for mortar and concrete, and careful tests made of the materials for the purpose of determining their qualities and the grading necessary to secure maximum density* or a minimum percentage of voids.

Fine Aggregate. 45. Fine aggregate consists of sand, crushed stone, or gravel screenings, passing when dry a screen having ¼-in. diameter holes. It should be preferably of silicious material, clean, coarse, free from vegetable loam or other deleterious matter.

46. A gradation of the grain from fine to coarse is generally advantageous.

47. Mortars composed of one part Portland cement and three parts fine aggregate by weight when made into briquettes should show a tensile strength of at least 70% of the strength of 1:3 mortar of the same consistency made with the same cement and standard Ottawa sand.

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* A convenient coefficient of density is the ratio of the sum of the volumes of materials contained in a unit volume to the total unit volume.

48. Coarse aggregate consists of inert material, such as crushed stone or gravel, which is retained on a screen having ¼-in. diameter holes. The particles should be clean, hard, durable, and free from all deleterious material. Aggregates containing soft, flat or elongated particles should be excluded from important structures. A gradation of sizes of the particles is generally advantageous. **Coarse Aggregate.**

49. The maximum size of the coarse aggregate shall be such that it will not separate from the mortar in laying and will not prevent the concrete from fully surrounding the reinforcement and filling all parts of the forms. Where concrete is used in mass, the size of the coarse aggregate may be such as to pass a 3-in. ring. For reinforced members a size to pass a 1-in. ring, or a smaller size, may be used.

50. Where cinder concrete is permissible, the cinders used as the coarse aggregate should be composed of hard, clean, vitreous clinker, free from sulphides, unburned coal, or ashes. **Cinders.**

51. The water used in mixing concrete should be free from oil, acid, strong alkalis, or vegetable matter. **Quality of Water.**

Steel.

52. The steel used for reinforcement shall be of the same quality as specified for structural steelwork in buildings. **Quality of Steel.**

53. Steel wire used for reinforcement should be drawn from rods of basic open-hearth steel of the same quality as that specified for rivet steel. **Wire.**

54. All steel to be embedded in concrete shall conform to the shape and sections shown on drawings, and shall be delivered unpainted. It shall be thoroughly cleansed from scale, grease, oil and rust, and given a coating of Portland cement grouting before being covered with concrete. The cleaning of the metal shall be done with suitable scrapers, steel brushes or such other tools as may most efficiently clean the surface.

Concrete.

55. The materials to be used in concrete shall be carefully selected, of uniform quality, and proportioned with a view to securing as nearly as possible a maximum density.

56. The unit of measure shall be the barrel, which should be taken as containing 3.8 cu. ft. Four bags containing 94 lb. of cement each shall be considered the equivalent of one barrel. Fine and coarse aggregate should be measured separately as loosely thrown into the measuring receptacle. **Unit of Measure.**

Relation of Fine and Coarse Aggregates. 57. The fine and coarse aggregates shall be used in such relative proportions as will insure maximum density.

Relation of Cement and Aggregates. 58. For reinforced concrete construction, a density proportion based on 1:6 should generally be used, *i. e.,* one part of cement to a total of six parts of fine and coarse aggregates measured separately.

59. In columns, richer mixtures are often required, while for massive masonry or rubble concrete a leaner mixture, of 1:9 or even 1:12, may be used.

Mixing. 60. The ingredients of concrete should be thoroughly mixed to the desired consistency, and the mixing should continue until the cement is uniformly distributed and the mass is uniform in color and homogeneous, since the maximum density and, therefore, the greatest strength of a given mixture depends largely on thorough and complete mixing.

Measuring Ingredients. 61. Methods of measurement of the proportions of the various ingredients, including the water, should be used, which will secure separate uniform measurements at all times.

Machine Mixing. 62. When the conditions will permit, a machine mixer of a type which insures the uniform proportioning of the materials throughout the mass should be used, since a more thorough and uniform consistency can be thus obtained.

Hand Mixing. 63. When it is necessary to mix by hand, the mixing should be on a water-tight platform, and especial precautions should be taken to turn the materials until they are homogeneous in appearance and color.

Consistency. 64. The materials shall be mixed wet enough to produce a concrete of such a consistency as will flow into the forms and about the metal reinforcement, and, at the same time, can be conveyed from the mixer to the forms without separation of the coarse aggregate from the mortar.

Retempering. 65. Retempering mortar or concrete, *i. e.,* remixing with water after it has partially set, shall not be permitted.

Placing of Concrete. 66. Concrete, after the addition of water to the mix, should be handled rapidly, and in as small masses as is practicable, from the place of mixing to the place of final deposit, and under no circumstances shall concrete be used that has partially set before final placing. A slow-setting cement should be used when a long time is likely to occur between mixing and final placing.

67. The concrete should be deposited in such a manner as will permit the most thorough compacting, such as can be obtained by

working with a straight shovel or slicing tool kept moving up and down until all the ingredients have settled in their proper place by gravity and the surplus water has been forced to the surface.

68. In depositing the concrete under water, special care should be exercised to prevent the cement from being floated away, and to prevent the formation of laitance, which hardens very slowly and forms a poor surface on which to deposit fresh concrete. Laitance is formed in both still and running water, and should be removed before placing fresh concrete.

69. Before placing the concrete, care should be taken to see that the forms are substantial and thoroughly wetted and the space to be occupied by the concrete is free from debris. When the placing of the concrete is suspended, all necessary grooves for joining future work should be made before the concrete has had time to set.

70. When work is resumed, concrete previously placed should be roughened, thoroughly cleansed of foreign material and laitance, drenched and slushed with a mortar consisting of one part Portland cement and not more than two parts fine aggregate.

71. The faces of concrete exposed to premature drying should be kept wet for a period of at least seven days.

Freezing Weather.

72. Concrete for reinforced structures should not be mixed or deposited at a freezing temperature, unless special precautions are taken to avoid the use of materials containing frost or covered with ice crystals, and to provide means to prevent the concrete from freezing after being placed in position and until it has thoroughly hardened.

Rubble Concrete.

73. Where the concrete is to be deposited in massive work, its value may be improved and its cost materially reduced through the use of clean stones thoroughly embedded in the concrete as near together as is possible and still entirely surrounded by concrete.

Forms and Supports.

74. The forms must have sufficient resistance to bending, as well as to shocks and vibrations due to tamping, and they shall be arranged to be safely removable while their supports are left in place. The forms should be as nearly watertight as possible, to prevent the escaping of the cement.

Removal of Forms.

75. In removing the forms and supports, all jar and vibration shall be avoided. No forms shall be removed except in the presence of the inspector. After the forms are removed, no patching or plastering shall be done until all surfaces have been inspected and permission given by the engineer.

76. The period which must elapse between the completion of the tamping and the removal of the forms is a matter of judgment and depends upon the weather, the distance between supports, and the weight of the parts of the structure. The side forms of beams and columns, and the forms of floor slabs up to spans of 5 ft. may be removed after the concrete has hardened sufficiently, that is, in a few days, while the supports of beams should not be removed in less than 14 days. For longer spans and larger sections, 4 to 6 weeks may be necessary.

77. In buildings of several stories, the supports of the lower floors shall not be removed until the hardening of the concrete is so far advanced that it can safely carry the load.

Protection of the Structure.

78. Immediately after the completion of the tamping, the structural parts shall be protected against the effect of freezing and premature drying, as well as against vibrations and loads, until the concrete is sufficiently hardened.

Inspection and Tests.

Facilities for Inspection.

79. All facilities for inspection of material and workmanship shall be furnished by the contractor to the engineer and his inspectors, who shall have free access to any part of the structure during construction, or to any part of the works in which any part of the material is made.

80. Inspection during construction shall cover the following:

a. The materials.

b. The correct construction and erection of the forms and supports.

c. The sizes, shapes and arrangement of the reinforcement.

d. The proportioning, mixing and placing of the concrete.

e. The strength of the concrete by tests of standard test pieces made on the work.

f. Whether the concrete is sufficiently hardened before the forms and supports are removed.

g. Prevention of injury to any part of the structure by and after the removal of the forms.

h. Comparison of dimensions of all parts of the finished structure with the plans.

Tests of Concrete.

81. Samples of concrete shall be taken from the wheelbarrows as it is being transported to the forms and tested in 8-in. cylinders, 16 in. long, to ascertain the crushing strength, as directed by the engineer.

82. All steel shall be tested before it is shipped from the mills, and all manufactured steel work inspected in the shops where the work is being done before shipment, as specified for structural steel work. **Tests of Steel.**

83. Load tests on portions of the finished structure shall be made where there is reasonable suspicion that the work has not been properly performed, or that, through influences of some kind, the strength has been impaired. A test load of twice the live load shall cause no permanent deformations. Load tests shall not be made until after 60 days of hardening. **Load Tests.**

FORMULAS FOR APPROXIMATE COMPUTATIONS RECOMMENDED BY THE GERMAN CONCRETE ASSOCIATION.

SIMPLE BENDING.

1. Rectangular Beams.

(*a*) Reinforced for tension only (see Fig. 1).

If A_s = total area of the reinforcement, in sq. in.

b = width of the beam in inches.

h = effective depth, $n = \frac{E_s}{E_c} = \begin{cases} 15 \text{ for stone concrete,} \\ 30 \text{ for cinder concrete.} \end{cases}$

M = moment of the exterior forces, in inch-pounds.

V = total vertical shear, in pounds.

Distance of neutral axis from top of beam

$$x = \frac{n A_s}{b}\left[-1 + \sqrt{1 + \frac{2bh}{nA_s}}\right]$$

Max. unit stress on concrete $\sigma_c = \dfrac{2M}{bx\left(h - \frac{x}{3}\right)}$

Unit stress on steel....... $\sigma_s = \dfrac{M}{A_s\left(h - \frac{x}{3}\right)}$

Unit shear.............. $\tau_0 = \dfrac{V}{b\left(h - \frac{x}{3}\right)}$

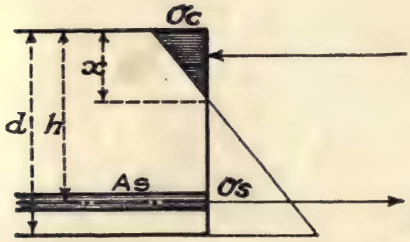


FIG 1

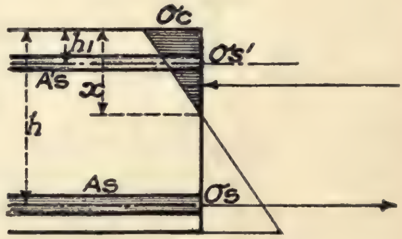


FIG 2

Unit bond stress on the reinforcing bars

$$\tau_1 = \frac{b\,\tau_0}{\text{Sum of perimeters of all bars}}.$$

A computation of the shear and bond for freely supported beams is not generally necessary.

(*b*) With double reinforcement for tension and compression (see Fig. 2).

The distance of neutral axis from top of beam

$$x = \frac{2n}{b}(h A_s + h' A'_s)$$

and the maximum unit compression on the concrete

$$\sigma_c = \frac{6 M x}{b x^2 (3h - x) + 6 A'_s n (x - h')(h - h')}.$$

Unit stress in tension in the lower reinforcement

$$\sigma_s = \frac{\sigma_c (h - x) n}{x}$$

Unit stress in compression on the upper reinforcement

$$\sigma'_s = \frac{\sigma_c (x - h') n}{x}$$

2. Beams of T Section.

The effective width b of the slab is to be assumed as $b \overline{>} \frac{1}{3} l$, where l denotes the effective length of the beam; b should, however, not be larger than the distance between stems.

For T beams, two cases have to be considered:

(*a*) When the neutral axis lies in the slab, or $x \overline{<} d$ (see Fig. 3).

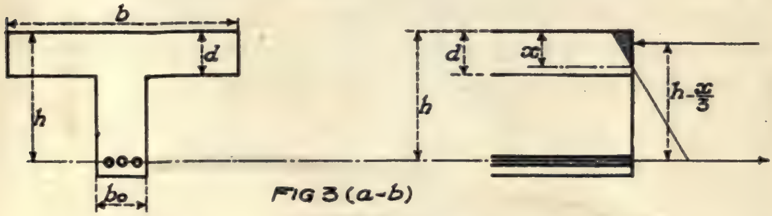


FIG 3 (a-b)

The formulas for rectangular beams reinforced for tension also apply to beams of T section when the shear in the stem and the bond in the reinforcement over the supports have to be computed.

(b) Where the neutral axis lies in the stem, or $x > d$ (see Fig. 4).

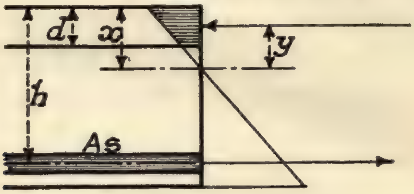


FIG 4

If we neglect the small compression in the stem of the beam, we get:

$$x = \frac{2\,n\,h\,A_s + b\,d^2}{2\,(n\,A_s + b\,d)} \text{ and } y = x - \frac{d}{2}\;\frac{d^2}{6\,(2\,x - d)}$$

$$\sigma_s = \frac{M}{A_s\,(h - x + y)} \text{ and } \sigma_c = \frac{\sigma_s\,x}{n\,(h - x)}.$$

COMPRESSION.

Columns in which buckling need not be considered.

(a) Axial pressure.

If A_c denotes the area of the concrete, the total safe load on the column

$$P = \sigma_c\,(A_c + n\,A_s), \text{ where } n = 15,$$

and

$$\sigma_c = \frac{P}{A_c + n\,A_s},\quad \sigma_s = \frac{P}{A_s + \frac{A_c}{n}} = n\,\sigma_c.$$

(b) Eccentric pressure (bending combined with axial pressure).

The computations can be made in the same manner as for sections of homogeneous material if, in the areas and moments of resistance, the section of the reinforcement is added to that of the concrete with $n = 15$ times its value. If tensile strains occur, the steel reinforcement on the tension side must be able to resist the same.

APPENDIX.

TABLE 1.—Weights of Building Materials, Etc., in Pounds per Cubic Foot.

Material.	Weight.	Material.	Weight.
Brick, pressed and paving	150	Hemlock	25
" common building	120	White pine	25
" soft building	100	Douglas fir	30
Granite	170	Yellow pine	40
Marble	170	White oak	50
Limestone	160	Mortar	100
Sandstone	150	Stone concrete	150
Cinders	40	Cinder "	110
Slag	160–180	Common brick work	100–120
Granulated furnace-slag	53	Rubble masonry, sandstone	130
Gravel	120	" " limestone	140
Slate	175	" " granite	150
Sand, clay and earth (dry)	100	Ashlar " sandstone	140
" " " " (moist)	120	" " limestone	150
Coal ashes	45	" " granite	165
Paving asphaltum	100	Masonry debris	90
Plaster of Paris	140	Cast iron	450
Glass	160	Wrought iron	480
Water	62½	Steel	490
Snow, freshly fallen	10	Lead	711
" wet	50	Copper, rolled	490
Spruce	25	Brass	523

Plaster, ceiling, 10 to 15 lb. per sq. ft.

TABLE 2.—Weights of Merchandise, Etc., Stored Loose in Heaps or Tanks, in Pounds per Cubic Foot.

Alcohol	52	Lime	60–80
Apples	47	Naphtha	50
Barley	40	Oats	30
Beans	55	Oils	55
Beets	40	Paper	35–60
Books	40	Peat, dry, unpressed	20–30
Canned Goods	45	Petroleum	55
Cement, natural	50–70	Pitch	75
" Portland	90–100	Potatoes	45
Chalk	156	Pumice Stone	56
Charcoal	15–30	Rags	20–45
Cheese	30	Rosin	68
Coal, soft	50	Rubber Goods	60–100
" hard	55	Salt, solid	134
Coke	30–50	" coarse	65
Cork	15	" fine table	80
Corn	38	Straw	10–20
Cotton Goods	40	Sugar	50
Fat	58	Sulphur	125
Flour	50	Tallow	59
Gunpowder	60	Tar	75
Gypsum	60–70	Tin, cast	462
Hay, loose	5	" in boxes	278
" baled	20	Wheat	50
Ice	55	Wines	62
Lard	59	Woolen Goods	25
Leather Goods	20		

If stored in bags, barrels, cases or boxes, multiply above given weights by 0.8, but take outside rectangular dimensions.

TABLE 3.—PERMISSIBLE COMPRESSIVE STRESSES FOR STEEL.

P = Stress allowed in lb. per sq. in.
l = Length in inches.
r = Least radius of gyration, in inches.

$$P = 16\,000 - 70\,\frac{l}{r}$$

$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P	$\frac{l}{r}$	P
28	14 000	60	11 800	92	9 560	124	7 320
30	13 900	62	11 660	94	9 420	126	7 180
32	13 760	64	11 520	96	9 280	128	7 040
34	13 620	66	11 380	98	9 140	130	6 900
36	13 480	68	11 240	100	9 000	132	6 760
38	13 340	70	11 100	102	8 860	134	6 620
40	13 200	72	10 960	104	8 720	136	6 480
42	13 060	74	10 820	106	8 580	138	6 340
44	12 920	76	10 680	108	8 440	140	6 200
46	12 780	78	10 540	110	8 300	142	6 060
48	12 640	80	10 400	112	8 160	144	5 920
50	12 500	82	10 260	114	8 020	146	5 780
52	12 360	84	10 120	116	7 880	148	5 640
54	12 220	86	9 980	118	7 740	150	5 500
56	12 080	88	9 840	120	7 600		
58	11 940	90	9 700	122	7 460		

TABLE 4.—Shearing and Bearing Value of Shop Rivets, in Pounds.

Dia. of Rivet, Inches. Fraction.	Decimal.	Area, in square inches.	Single shear at 12 000 lb.	Bearing Value for Different Thicknesses of Plate, in Inches, at 24 000 Pounds per Sq. In. ¼	5/16	⅜	7/16	½	9/16	⅝	11/16	¾	13/16	⅞	15/16	1
⅜	0.375	0.1104	1 320	2 250	2 810	3 380										
½	0.500	0.1963	2 360	3 000	3 750	4 500	5 250	6 000								
⅝	0.625	0.3068	3 680	3 750	4 690	5 630	6 560	7 500	8 440	9 380						
¾	0.750	0.4418	5 300	4 500	5 630	6 750	7 880	9 000	10 130	11 250	12 380	13 500				
⅞	0.875	0.6013	7 220	5 250	6 560	7 880	9 190	10 500	11 810	13 130	14 440	15 750	17 060	18 380		
1	1.000	0.7854	9 420	6 000	7 500	9 000	10 500	12 000	13 500	15 000	16 500	18 000	19 500	21 000	22 500	24 000

Shearing and Bearing Value of Field Rivets and Bolts, in Pounds.

Dia. of Rivet, Inches. Fraction.	Decimal.	Area, in square inches.	Single shear at 10 000 lb.	Bearing Value for Different Thicknesses of Plate, in Inches, at 20 000 Pounds per Sq. In. ¼	5/16	⅜	7/16	½	9/16	⅝	11/16	¾	13/16	⅞	15/16	1
⅜	0.375	0.1104	1 100	1 880	2 340	2 810										
½	0.500	0.1963	1 960	2 500	3 130	3 750	4 380	5 000								
⅝	0.625	0.3068	3 070	3 130	3 910	4 690	5 470	6 250	7 030	7 810						
¾	0.750	0.4418	4 420	3 750	4 690	5 630	6 560	7 500	8 440	9 380	10 310	11 250				
⅞	0.875	0.6013	6 010	4 380	5 470	6 560	7 660	8 750	9 840	10 940	12 030	13 130	14 220	15 310		
1	1.000	0.7854	7 850	5 000	6 250	7 500	8 750	10 000	11 250	12 500	13 750	15 000	16 250	17 500	18 750	20 000

Note.—All Bearing Values above or to right of upper Zigzag Lines are greater than Double Shear. Values below or to left of lower Zigzag Lines are smaller than Single Shear.

TABLE 5.—Maximum Bending Moments on Pins.

Extreme Fiber Stress of 24 000 Lb. per Sq. In.

Dia. of pin, in inches.	Area of pin, in sq. in.	Moments, in inch-pounds.	Dia. of pin, in inches.	Area of pin, in sq. in.	Moments, in inch-pounds.
2	3.142	18 850	6½	33.183	647 070
2⅛	3.547	22 610	6⅝	34.472	685 120
2¼	3.976	26 840	6¾	35.785	724 640
2⅜	4.430	31 560	6⅞	37.122	765 650
2½	4.909	36 820	7	38.485	808 170
2⅝	5.412	42 620	7⅛	39.871	852 250
2¾	5.940	49 000	7¼	41.282	897 890
2⅞	6.492	55 990	7⅜	42.718	945 140
3	7.069	63 620	7½	44.179	994 020
3⅛	7.670	71 910	7⅝	45.664	1 044 550
3¼	8.296	80 880	7¾	47.173	1 096 770
3⅜	8.946	90 580	7⅞	48.707	1 150 700
3½	9.621	101 020	8	50.265	1 206 370
3⅝	10.321	112 240	8⅛	51.849	1 263 810
3¾	11.045	124 250	8¼	53.456	1 323 040
3⅞	11.793	137 100	8⅜	55.088	1 384 090
4	12.566	150 800	8½	56.745	1 447 000
4⅛	13.364	165 380	8⅝	58.426	1 511 780
4¼	14.186	180 870	8¾	60.132	1 578 470
4⅜	15.033	197 310	8⅞	61.862	1 647 080
4½	15.904	214 710	9	63.617	1 717 660
4⅝	16.800	233 100	9⅛	65.397	1 790 230
4¾	17.721	252 520	9¼	67.201	1 864 820
4⅞	18.665	272 980	9⅜	69.029	1 941 360
5	19.635	294 520	9½	70.882	2 020 140
5⅛	20.629	317 170	9⅝	72.760	2 100 940
5¼	21.648	340 950	9¾	74.662	2 183 860
5⅜	22.691	365 890	9⅞	76.590	2 268 940
5½	23.758	392 010	10	78.54	2 356 190
5⅝	24.850	419 350	10¼	82.52	2 537 360
5¾	25.967	447 930	10½	86.59	2 727 590
5⅞	27.109	477 790	10¾	90.76	2 927 090
6	28.274	508 940	11	95.03	3 136 090
6⅛	29.465	541 410	11¼	99.40	3 354 810
6¼	30.680	575 240	11½	103.87	3 583 480
6⅜	31.919	610 450	12	113.10	4 071 500

TABLE 6.—Thickness of Spruce and White Pine Plank for Floors.

Span in feet.	Thickness of Plank in Inches for Various Loads per Sq. Ft.																
	lb. 30	lb. 40	lb. 50	lb. 75	lb. 100	lb 125	lb. 150	lb. 175	lb. 200	lb. 225	lb. 250	lb. 275	lb. 300	lb. 325	lb. 350	lb. 375	lb. 400
4	0.9	1.1	1.2	1.5	1.7	1.9	2.1	2.2	1.4	2.5	2.7	2.8	2.9	3.1	3.2	3.3	3.4
5	1.2	1.4	1.5	1.8	2.1	2.4	2.6	2.8	3.0	3.2	3.4	3.5	3.7	3.8	4.0	4.1	4.3
6	1.4	1.6	1.8	2.2	2.6	2.9	3.1	3.4	3.6	3.8	4.0	4.2	4.4	4.6	4.8	4.9	5.1
7	1.7	1.9	2.1	2.6	3.0	3.3	3.7	3.9	4.2	4.5	4.7	4.9	5.2	5.4	5.6	5.8	5.9
8	1.9	2.2	2.4	3.0	3.4	3.8	4.2	4.5	4.8	5.1	5.4	5.7	5.9	6.1	....	....	....
9	2.1	2.5	2.7	3.4	3.9	4.3	4.7	5.1	5.4	5.8	6.1	..	..	....	....	....	....
10	2.4	2.7	3.1	3.7	4.3	4.8	5.2	5.6	6.0	....	....	....	...	....	....	....	....
11	2.6	3.0	3.4	4.1	4.7	5.3	5.8	....	....	....	....	....	....	....	....	....	....
12	2.9	3.3	3.7	4.5	5.2	....	....	....	....	....	....	....	....	....	....	....	....
13	3.1	3.6	4.0	4.9	5.6	....	....	....	....	....	....	....	....	...	....	....	....
14	3.4	3.9	4.3	5.3	6.1	....	....	....	....	....	....	....	....	....	....	....	....

For Yellow Pine use nine-tenths of the above thickness.

TABLE 7

STANDARD DIMENSIONS FOR COLUMNS

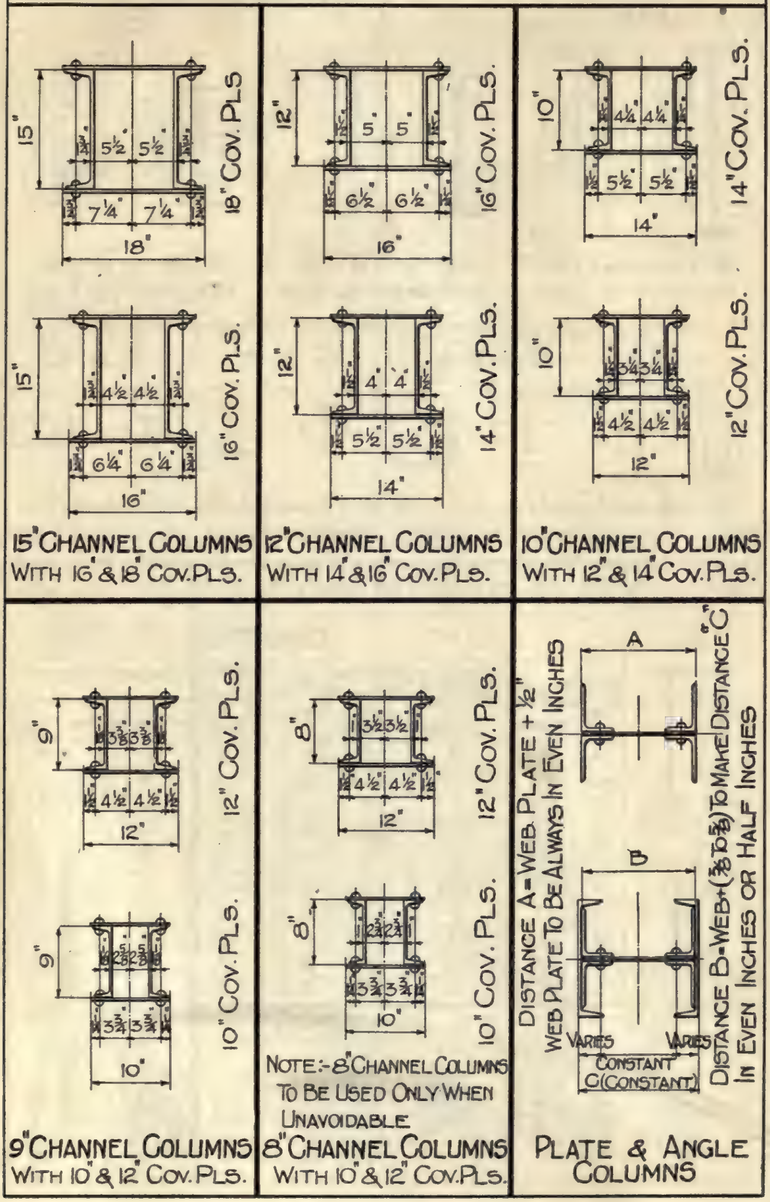
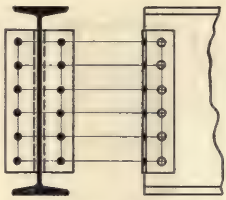


TABLE 8

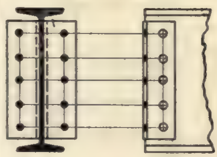
STANDARD FRAMING OF BEAMS

24"



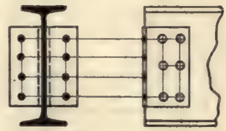
2 Ls 4×4×7/16×1'-6"
WEIGHT 41 LBS.

20" & 18"



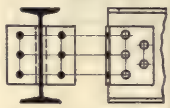
2 Ls 4×4×7/16×1'-3"
WEIGHT 34 LBS.

15"



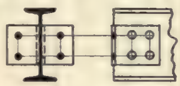
2 Ls 6×4×7/16×0'-10"
WEIGHT 29 LBS.

12"



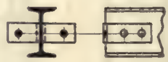
2 Ls 6×4×7/16×7½"
WEIGHT 22 LBS.

10", 9" 8" & 7"



2 Ls 6×4×7/16×0'-5"
WEIGHT 15 LBS.

6" & 5"

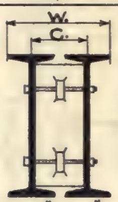


2 Ls 6×4×7/16×0'-2½"
WEIGHT 7 LBS.

4" & 3"

2 Ls 6×4×7/16×0'-2"
WEIGHT 6 LBS.

AMERICAN BRIDGE CO. STANDARD CAST IRON SEPARATORS

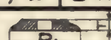


FOR 12" TO 24" Is

FOR 6" TO 10" Is

BEAMS				SEPARATORS	
SIZE INCHES	WEIGHT PER FT. POUNDS	DISTANCE CENTER TO CENTER C. INCHES	OUT TO OUT OF FLANGES W. INCHES	TOTAL WEIGHT	INCREASE IN WEIGHT OF SEPARATOR FOR 1" ADD'AL SPREAD OF BEAM
24	80	8	15	32.3	3.6
20	65	7	13¼	24.2	2.9
18	55	7	13	22.2	2.5
15	42	6½	12	15.0	1.6
12	31½	6	11	11.9	1.3
10	25	5½	10	8.4	1.1
9	21	5	9¼	6.3	.9
8	18	4½	8½	5.2	.8
7	15	4½	8¼	5.2	.7
6	12¼	4	7½	5.2	.6

CAST WASHERS



BOLT	D.	T.	WEIGHT
5/8	2¾	½	.41
¾	3¼	5/8	.7
7/8	3¾	¾	1.0

GAS PIPE SEPARATORS AND BOLTS

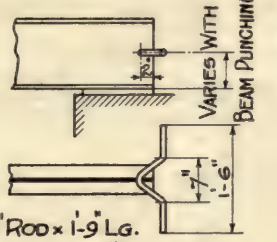
DISTANCE C. TO C. OF BEAMS	WEIGHT OF PIPE AND BOLT
2	1.2
3	1.4
4	1.6
5	1.9
6	2.2
7	2.5
8	2.7
9	2.9
10	3.2
11	3.5
12	3.8
13	4.0
14	4.2
16	4.8
18	5.3

WEIGHTS GIVEN ARE FOR 1" ϕ PIPE AND ¾" ϕ BOLT

TABLE 9

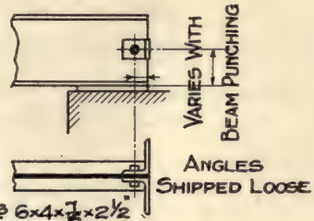
STANDARD DETAILS

GOVERNMENT ANCHOR



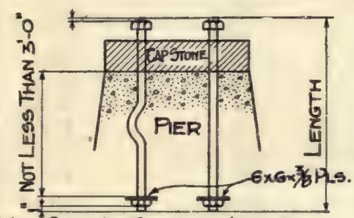
¾" ROD × 1'-9" LG.
WEIGHT 3#

ANGLE ANCHOR



2 Ls 6×4×⅞×2½"
1 BOLT ¾"φ
WEIGHT WITH BOLT 7#

BUILT IN ANCHOR BOLTS



WHEN BOLTS ARE SEPARATED LESS THAN
WIDTH OF WASHER USE WASHER WITH TWO HOLES

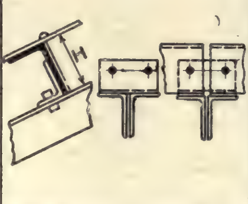
SWEDGE BOLTS

DIAM. INCHES	LENGTH FT. & INS.	WEIGHT INCLUDING NUT-LBS
¾	0'-9"	2
⅞	1'-0"	3
1	1'-0"	4
1¼	1'-3"	7

PURLIN CONNECTIONS

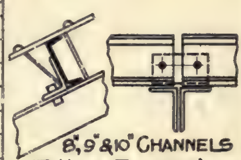
ANGLE PURLINS

LEGH OF PURLIN	CLIP ANGLE
2	1¾×1¾
2½	2×2
3	2½×2½ OR 2
3½	3×2½
4	3×2½
5	3½×2½



CHANNEL PURLINS

PURLIN	CLIP ANGLE
4	3×2½
5	3½×2½
6	4×3
7	4×3
8	4×3
9	5×3½
10	5×3½



8", 9" & 10" CHANNELS
TO HAVE FLANGE ALSO
ATTACHED TO RAFTERS

I BEAM PURLINS

PURLIN	CLIP ANGLE
4	3½×2½
5	4×3
6	4×3

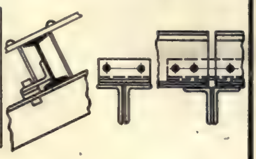
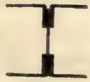


TABLE 10.—PLATE GIRDERS.



Tension 16 000 Lb. per Sq. In. Net. Area.

Each flange 2 angles.	Web plate.	Weight per ft.	Maximum moments, thousands of ft. lbs.
$3 \times 2\frac{1}{2} \times \frac{1}{4}$	$24 \times \frac{1}{4}$	38.4	90
$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{1}{4}$	$24 \times \frac{1}{4}$	40.0	99
	$30 \times \frac{1}{4}$	45.1	132
$3\frac{1}{2} \times 2\frac{1}{2} \times \frac{5}{16}$	$30 \times \frac{1}{4}$	49.9	154
$4 \times 3 \times \frac{5}{16}$	$30 \times \frac{1}{4}$	54.3	177
	$36 \times \frac{1}{4}$	59.4	222
$5 \times 3\frac{1}{2} \times \frac{5}{16}$	$36 \times \frac{1}{4}$	65.4	265
	$42 \times \frac{1}{4}$	70.5	320
$5 \times 3\frac{1}{2} \times \frac{3}{8}$	$36 \times \frac{5}{16}$	79.8	318
	$42 \times \frac{5}{16}$	86.2	385
	$48 \times \frac{5}{16}$	92.6	457
$6 \times 4 \times \frac{3}{8}$	$36 \times \frac{5}{16}$	87.4	367
	$42 \times \frac{5}{16}$	93.8	443
	$48 \times \frac{5}{16}$	100.2	524
$6 \times 4 \times \frac{7}{16}$	$36 \times \frac{5}{16}$	95.4	415
	$42 \times \frac{5}{16}$	101.8	500
	$48 \times \frac{5}{16}$	108.2	589
$6 \times 4 \times \frac{1}{2}$	$36 \times \frac{5}{16}$	103.0	462
	$42 \times \frac{5}{16}$	109.4	555
	$48 \times \frac{5}{16}$	115.8	652
	$60 \times \frac{5}{16}$	128.6	856
$6 \times 4 \times \frac{5}{8}$	$36 \times \frac{3}{8}$	125.9	565
	$42 \times \frac{3}{8}$	133.6	679
	$48 \times \frac{3}{8}$	141.2	797
	$60 \times \frac{3}{8}$	156.6	1 046
$6 \times 6 \times \frac{1}{2}$	$36 \times \frac{3}{8}$	124.3	544
	$48 \times \frac{3}{8}$	139.6	774
	$60 \times \frac{3}{8}$	155.0	1 023
$6 \times 6 \times \frac{5}{8}$	$36 \times \frac{3}{8}$	142.7	652
	$48 \times \frac{3}{8}$	158.0	923
	$60 \times \frac{3}{8}$	173.4	1 211
$6 \times 6 \times \frac{3}{4}$	$36 \times \frac{3}{8}$	176.0	781
	$48 \times \frac{3}{8}$	176.0	1 066
	$60 \times \frac{3}{8}$	191.4	1 393
	$72 \times \frac{3}{8}$	206.6	1 739

TABLE 11.—Weights of Roof Trusses and Purlins for Uniform Loadings.

Weight of Trusses.

$$W = \frac{P\,L}{300 + 6\,L + \dfrac{P\,D}{3}},$$

in which

W = weight of truss per square foot of building;
L = span of truss in feet;
D = distance center to center of trusses in feet;
P = load per square foot on truss.

Weight of Purlins.

$$W_1 = \frac{\sqrt{P_1\,D}}{45} - \frac{1}{4},$$

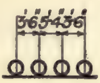
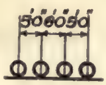
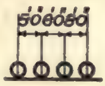
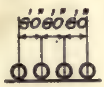
in which

W_1 = weight of purlins per square foot of building;
D = distance center to center of trusses in feet;
P_1 = load per square foot on purlins.

TABLE 12.—Typical Hand Cranes.

Capacity in tons.	Span.	Wheel base. 	Maximum wheel load in pounds.	Side clearance.	Vertical clearance.	Weight of Rail for: Plate Girders.	Beams.
2	30	4 ft. 0 in.	3 100	7 in.	4 ft. 0 in.	30 lbs. per yd.	30
	50	5 " 0 "	4 000	7 "	4 " 0 "	30 "	30
4	30	4 " 0 "	5 400	8 "	4 " 6 "	30 "	30
	50	5 " 0 "	6 500	8 "	4 " 0 "	30 "	30
6	30	6 " 0 "	8 000	9 "	5 ft. 0 "	35 "	30
	50	7 " 0 "	9 200	9 "	5 " 0 "	35 "	30
8	30	6 " 0 "	10 500	10 "	5 " 0 "	40 "	35
	50	7 " 0 "	11 800	10 "	5 " 0 "	40 "	35
10	30	7 " 0 "	13 000	10 "	5 " 0 "	40 "	40
	50	8 " 0 "	14 400	10 "	5 " 6 "	40 "	40
16	30	7 " 0 "	20 700	10 "	5 " 6 "	45 "	45
	50	8 " 0 "	22 300	10 "	5 " 6 "	45 "	45
20	30	7 " 0 "	26 000	10 "	5 " 6 "	50 "	50
	50	8 " 0 "	28 000	10 "	5 " 6 "	50 "	50
25	30	7 " 0 "	32 300	12 "	6 " 0 "	55 "	50
	50	8 " 0 "	35 000	12 "	6 " 0 "	55 "	50

TABLE 13.—TYPICAL ELECTRIC TRAVELING CRANES.

Capacity, in tons.	Span.	Wheel base.	Maximum wheel load, in pounds.	Side clearance.	Vertical clearance.	WEIGHT OF RAIL FOR:	
						Plate girders.	Beams.
5	40	8 ft. 6 in.	12 000	10 in.	6 ft. 0 in.	40 lb. per yd.	40
	60	9 " 0 "	13 000	10 "	6 " 0 "	40 "	40
10	40	9 " 0 "	19 000	10 "	6 " 0 "	45 "	40
	60	9 " 6 "	21 000	10 "	6 " 0 "	45 "	40
15	40	9 " 6 "	25 000	10 "	7 " 0 "	50 "	50
	60	10 " 0 "	29 000	10 "	7 " 0 "	50 "	50
20	40	10 " 0 "	33 000	12 "	7 " 0 "	55 "	50
	60	10 " 6 "	36 000	12 "	7 " 0 "	55 "	50
25	40	10 " 0 "	40 000	12 "	8 " 0 "	60 "	50
	60	10 " 6 "	44 000	12 "	8 " 0 "	60 "	50
30	40	10 " 6 "	48 000	12 "	8 " 0 "	70 "	60
	60	11 " 0 "	52 000	12 "	8 " 0 "	70 "	60
40	40	11 " 0 "	64 000	12 "	9 " 0 "	80 "	60
	60	12 " 0 "	70 000	12 "	9 " 0 "	80 "	60
50	40	11 " 0 "	72 000	14 "	9 " 0 "	100 "	60
	60	12 " 0 "	80 000	14 "	9 " 0 "	100 "	60
60	40	13 " 0 "	88 000	16 "	9 " 0 "	100 "	....
	60	14 " 0 "	94 000	16 "	9 " 0 "	100 "	....
	80	15 " 0 "	103 000	16 "	9 " 0 "	100 "	....
60	40	....	44 000	16 "	10 " 6 "	100 "	....
	60		47 000	16 "	10 " 6 "	100 "	....
	80		51 500	16 "	10 " 6 "	100 "	....
75	40	....	55 000	16 "	12 " 0 "	100 "	....
	60		60 000	16 "	12 " 0 "	100 "	....
	80		64 000	16 "	12 " 0 "	100 "	....
100	40	....	83 000	19 "	13 " 6 "	100 "	....
	60		86 000	19 "	13 " 6 "	100 "	....
	80		89 000	19 "	13 " 6 "	100 "	....
150	40	....	130 000	20 "	16 " 0 "	150 "	....
	60		134 000	20 "	16 " 0 "	150	....
	80		139 000	20 "	16 " 0 "	150 "	....

Loads, clearances, etc., given above are for end carriages of electric cranes.

ABSTRACTS FROM THE FOLLOWING BUILDING LAWS:

In the following tables:

l = Unsupported length, in inches;

l_1 = Effective " " "

r = Corresponding radius of gyration, in inches;

d = Least lateral dimension, in inches;

W = Total load, in lb., uniformly distributed.

TABLE

MINIMUM LIVE LOADS FOR

Structure.	POUNDS PER					
	New York, 1906.	Chicago, 1905.	Philadelphia, 1907.	Baltimore, 1908.	St. Louis, 1907.	Boston, 1907.
Dwellings, apartment houses, hotels, etc.	60	Dwellings, apartment houses, 40; hotels, etc., 50.	70	60	60	50 For room > 500 sq. ft., 100.
Office buildings, first floor	150	50	100	150	150	100
Office buildings, above first floor	75	50	100	75	70	100
Public assembly rooms, churches, theatres, etc.	90	100	120	75 with, 125 without, fixed seats.	100	200
Schools or places of instruction	75	75	....	75	100	Assembly rooms, 125; other rooms, 60.
Machine shops, armories, drill-rooms, etc.	....	....	....	....	....	....
Light manufacturing and retail stores and storehouses	120, not including machinery.	100	120	125	150	125
Heavy storehouses, warehouses and factories	150, not including machinery.	100	150	Factory, 175; storage, 250.	150	250
Stables or carriage houses	75	Area < 500 sq. ft., 40; larger floors, 100.	....	100	....	....
Stairways	....	....	....	....	....	70
Sidewalks	300	....	....	200	....	....
Roofs, per square foot of superficial surface	For slopes < 20°, 50.	....	30	....	....	....
Roofs, per square foot of horizontal projection	For slopes > 20°, 30.	25	....	For slopes > 20°, 20. < 20°, 40.	For flat roofs, 40.	For flat roofs, 40.
Wind, per square foot of elevation	30	When h't is = $1\frac{1}{2}$ width, 30.	35, reduced. See Building Laws, pp. 32–33.	30	30	....

14 A.

FLOORS, ROOFS, AND WALLS.

SQUARE FOOT.

Buffalo, 1905.	San Francisco, 1907.	Milwaukee. 1901.	District of Columbia, 1902.	Minneapolis, 1907.	Providence. 1905.	Rochester, 1904.	New Haven, 1905.	Schneider's, 1910.
Dwellings, 40; apart. houses, hotels, etc., 70	First floor 150; other floors, 75.	40	Halls, dining rooms, offices, etc., 75; other rooms, 50.	50	70 except attic.	50	70	40 * 2 000 † 500 ‡
70	150	60	Halls and Lobbies, 1:0; other floor space, 75.	150	150	70	100	80 * 5 000 † 1 000 ‡
70	75	60	do.	75	150	70	100	50 * 5 000 † 1 000 ‡
100	125	80	110	125	125	Theaters, 80 others, 70	120	100 * 5 000 † 1 000 ‡
100	75	50	75	100	100	70	70	60 * 5 000 † 1 000 ‡
....	....	250	....	....	250	....	....	....
120	120; not including machinery.	100	110	100	150	100	150	80 * 8 000 † 1 000 ‡
150	250; not including machinery.	....	200	200	250	150; increase for machinery.	150	120 up * special † special ‡
Public, 120; Private, 40.	75	....	200	85	70	Public, 100; Private, 50.	..	80 * 8 000 † 1 000 ‡
....	....	100; lower supports to carry two-thirds of total wt.	....	....	....	....	....	....
....	300	....	....	300	....	....	....	300 * 10 000 † 1 000 ‡
....	For slopes < 20°, 50.	30	25	....	50	....	40	Flat roofs, 40 * 2 000 † 500 ‡ slopes, special.
40	For slopes < 20°, 30.	....	....	30	....	40	....	
When h't is = 1½ width, 30	30	30 at twelfth story. 2½ less at each lower story.	30	....	....	30	....	30

* Uniform load in pounds per square foot of floor area.
† Concentrated load in pounds, which shall be applied to any point of the floor.
‡ Uniform load, in pounds per linear foot, for girders.

TABLE 14 B.

Permissible Reduction of Live Loads under Footings of Foundations in Buildings More than Three Stories High.

New York, 1906.	Philadelphia, 1907.	Baltimore, 1908.	St. Louis, 1907.	Boston, 1907.	San Francisco, 1907.	District of Columbia, 1902.	Schneider's, 1910.
For warehouses and factories, no reduction. For stores and buildings for light manufacturing, churches, schoolhouses and places of public assembly and amusement, reduction of 25%. For office buildings, hotels, dwelling houses, apartment houses, tenement houses, lodging houses and stables, reduction of 40%.	For light manufacturing buildings, reduction of $x = 100 - \frac{2}{5}\sqrt{\text{area}}$. For apartment houses, office buildings and hospitals, $x = 100 - \frac{4}{5}\sqrt{\text{area}}$ x = % of live load.	For warehouses, reduction of 50%. For other buildings, reduction of 75%.	Live load per sq. ft. on all floors, reduced to 10 lb. for office buildings and tenement houses, 20 lb. For mercantile buildings and 40 lb. for warehouses.	For warehouses, heavy mercantile and manufacturing buildings, no reduction. Other buildings, live load reduced for 2 floors, 15% " 3 " 20% " 4 " 25% " 5 " 30% " 6 " 35% " 7 " 40% " 8 " 45% " 9 or more, 50%	For warehouses, stores, libraries, halls and theaters, no reduction. For offices, dwellings, apartment houses, hotels, lodging houses, hospitals and schools, live load reduced 40%.	For warehouses and factories, no reduction. For stores and buildings for light manufacturing purposes, reduction of 25%. For churches, schoolhouses and places of public assembly, reduction of 10%. For office buildings, apartment houses, tenement houses, stables, etc., reduction of 40% if of masonry construction; of 25% if of steel skeleton construction.	With more than 5 floors, for each succeeding floor below the top floor, total live load reduced 5% until 50% is reached. All footings have same proportional area for dead load as the footing receiving largest ratio of live to dead load. For warehouses, no reduction.

TABLE 14 C.

PERMISSIBLE REDUCTION OF LIVE LOADS ON COLUMNS IN BUILDINGS MORE THAN THREE STORIES HIGH.

New York, 1906.	Philadelphia, 1907.	Baltimore, 1908.	St. Louis, 1907.	Boston, 1907.	San Francisco, 1907.	Minneapolis, 1900.	New Haven. 1905.	Schneider's, 1910.
Roof and top floor, no reduction. Each succeeding floor reduced 5% until 50% is reached.	For light manufacturing buildings, reduction of $x = 100 - \frac{2}{5}\sqrt{\text{area}}$. For apartment houses, office buildings, and hospitals, $x = 100 - \frac{4}{5}\sqrt{\text{area}}$. x = % of live load.	For cols. supporting roof and top floor, no reduction. For cols. supporting each succeeding floor, total live load reduced 5% until 50% is reached.	Floor above col. full live load. Each succeeding floor load reduced 5% until 40% is reached.	For warehouses. heavy mercantile and manufacturing buildings, no reduction. Other buildings, live load reduced for 2 floors, 15%; " 3 " 20%; " 4 " 25%; " 5 " 30%; " 6 " 35%; " 7 " 40%; " 8 " 45%; " 9 or more, 50%	For warehouses. stores, libraries, halls and theaters, no reduction. For offices, dwellings, apartment houses, hotels, lodging houses, hospitals and schools, live load reduced 40%.	For cols. supporting roof and top floor, no reduction. For cols. supporting each succeeding floor, total live load reduced 5% until 50% is reached.	No reduction.	For cols. supporting roof and top floor, no reduction. For cols. supporting each succeeding floor, total live load reduced 5% until 50% is reached. For warehouses, no reduction.

PERMISSIBLE REDUCTION OF LIVE LOADS ON GIRDERS.

New York, 1906.	Philadelphia, 1907.	Baltimore, 1908.	St. Louis, 1907.	Boston, 1907.	San Francisco, 1907.	Minneapolis, 1900.	New Haven. 1905.	Schneider's, 1910.
	For light manufacturing buildings. reduction of $x = 100 - \frac{2}{5}\sqrt{\text{area}}$. For apartment houses, office buildings and hospitals, $x = 100 - \frac{4}{5}\sqrt{\text{area}}$. x = % of live load.		For rolled beams. no reduction. For riveted girders, reduction of 15%.	For girders carrying more than 100 sq. ft., live load reduced 10%.	For rolled beams, no reduction. For girders in warehouses, stores, libraries. halls and theaters, no reduction. For girders in offices, dwellings. apartment houses, hotels, lodging houses, hospitals and schools, reduction of 20%.			

TABLE 14 D.

Bearing Capacity of Different Kinds of Soils.

Bearing material.	New York, 1906.	Chicago, 1905.	Philadelphia, 1907.	Baltimore, 1908.	St. Louis, 1907.	Buffalo, 1905.	San Francisco, 1907.	Milwaukee, 1901.	Dist. of Columbia, 1902.	Minneapolis, 1907.	Rochester, 1904.	New Haven, 1905.	Schneider's 1910.
	Tons per Square Foot.												
Soft clay	1		*	1	According to test with a maximum for any soil of 8 tons per sq. ft.	*	1	1	1		1		1
Dry clay		1¾	3½			3½	3	2		3½			
Hard clay				4	8	3½	4	4					
Stiff gravel or hard clay	4		3½			3½	4 to 6		4		6	4	
Hard pan				12		3½					10		
Clay and sand in layers, wet	2		*	2		*	2		2		2		
Clay and sand in layers, dry		1½		3		*							2
Loam clay or fine sand, firm and dry	3		*	3		*	3		3		3	4	
Dry clay and dry sand						*							3
Hard clay and firm, coarse sand				4		*							4
Very firm, coarse sand	4		3½			*	4	4	4		6	4	
Dry sand		2				*		2	3	3½			
Gravel and sand (well cemented)			6			*		8					
Firm, coarse sand and gravel				6		*							6
Concrete in foundations		12½				4			8 to 9	4			
Dimension stones in foundations		10				6				6			
Dressed foundation stones, in cement mortar						7				7			
	Tons.												
Maximum pressure on one timber pile	20†	25	20				20	†	25†	25†	20†		20 tons or 600 lb. per sq. in. of cross-sec.

* Consult Department of Public Works. † Safe sustaining power of one pile, in tons, is $\frac{2 \times \text{weight of hammer in tons} \times \text{height of fall in feet}}{\text{penetration, in inches (under last blow)} + 1}$.

TABLE 14 E.

Bearing Capacity of Materials.

Bearing material.	Tons per Square Foot. New York, 1906.	Chicago, 1905.	Philadelphia, 1907.	Baltimore, 1908.	St. Louis, 1907.	Boston, 1907.	Buffalo, 1905.	San Francisco, 1907.	Milwaukee, 1901.	Dist. of Columbia, 1902.	Minneapolis, 1907.	Rochester, 1904.	New Haven, 1905.	Schneider's, 1910.
Concrete: Portland cement 1, sand 2, stone 4	16½	12½	15	28.8	......	30	4	16½	......	16½	16½	16½	11	25
Concrete: Portland cement 1, sand 2, stone 5	15	12½	15	25.2	......	......	4	......	......	15	15	15	11	20
Concrete: Portland cement 1, sand 3, stone 5	......	......	......	......	18	......	......	......	......	......	15	......	......	......
Rubble in Portland cement mortar	10	......	10	9	......	......	5	......	......	10	10	10	......	10 to 12
Brickwork in Portland cement mortar	18	12½	15	18	21.6	20	12	......	18	18	18	18	15	12 to 15
Granite	72 to 173	......	......	72 to 173	......	60	*	27	......	72 to 173	72 to 144	72 to 173	......	30
Marble and limestone	48 to 166	......	......	72 to 144	......	40	*	......	......	48 to 166	48 to 166	48 to 166	......	25
Sandstone	29 to 115	......	......	......	......	30	*	......	......	29 to 115	29 to 115	29 to 115	......	20
Bluestone	144	......	......	......	......	......	......	......	......	144	......	......	......	25
Slate	72	......	......	......	......	......	......	......	......	72	......	72	......	......

* Use best practice.

TABLE 14 F.

PERMISSIBLE UNIT STRESSES—IRON, STEEL, AND WOOD.

Material.	New York, 1906.	Chicago, 1905.	Philadelphia, 1907.	Baltimore, 1908.	St. Louis, 1907.	Boston, 1907.	Buffalo, 1905.	San Francisco, 1907.	Milwaukee, 1901.	Dist. of Columbia, 1902.	Minneapolis, 1907.	Rochester, 1904.	New Haven, 1905.	Schneider's 1910.
	Pounds per Square Inch.													
Tension:														
Rolled steel	16 000	15 000	*16 250	16 000		16 000	16 000	16 000	16 000	16 000	16 000	16 000	16 000	16 000
Cast "	16 000			16 000				16 000		16 000	16 000	16 000		16 000
Cast iron	3 000	2 500		5 000		3 000		3 000	3 000	3 000	3 000	3 000		2 500
Yellow pine	1 200		1 800	1 800						1 200	1 200	1 200		
White "	800			1 000				700		800	800	800		
Spruce	800		1 250	1 200						800	800	800		
Oak	1 000			1 500						1 000	1 000	1 000		
Hemlock	600		1 000	800								600		
Compression:														
Rolled steel	16 000	15 000	*16 250	16 000	12 000	16 000		16 000		16 000	16 000	16 000	16 000	16 000
Cast "	16 000			16 000				16 000		16 000	16 000	16 000		16 000
Cast iron	16 000		16 700	16 000				16 000		16 000	16 000	16 000		12 000
Steel pins and shop rivs. (Brg.)	20 000	20 000	22 000	20 000		18 000	15 000	20 000		20 000	18 000	20 000	15 000	24 000
Oak (with grain)	900			1 000						900		900		1 200
" (across grain)	800			600						800		800		500
Yellow pine (with grain)	1 000		750	1 000						1 000		1 000		1 500
" " (across ")	600		92	600		500				600		600		350
White " (with ")	800			800				1 100		800		800		1 000
" " (across ")	400			400						400		400		200
Spruce (with ")	800		500	800						800		800		1 000
" (across ")	400		50	400		250				400		400		200
Locust (with ")	1 200			1 200						1 200		1 200		
" (across ")	1 000			1 000						1 000		1 000		
Hemlock (with ")	500		350	600								500		800
" (across ")	500		40	500								500		200
Chestnut (with ")	500									500		500		
" (across ")	500									1 000		1 000		

* Soft steel = 14 500.

TABLE 14 F (*continued*).

Permissible Unit Stresses—Iron, Steel, and Wood.

Pounds per Square Inch.

Material.	New York, 1906.	Chicago, 1905.	Philadelphia, 1907.	Baltimore, 1908.	St. Louis, 1907.	Boston, 1907.	Buffalo, 1905.
Columns: Steel $\frac{l}{r}$ = 40. 60. 80. 100. 120. 140. 160. 180. 200. 220. 240.	$15\,200 - 58\frac{l}{r}$	15 000 reduced by approved formulæ.	$\frac{16\,250}{1+\frac{l^2}{11\,000\,r^2}}$ for medium steel. $\frac{14\,500}{1+\frac{l^2}{13\,500\,r^2}}$ for soft steel.	$\frac{15\,000}{1+\frac{l^2}{13\,500\,r^2}}$ for medium steel. $\frac{14\,000}{1+\frac{l^2}{13\,500\,r^2}}$ for soft steel.	12 444 11 262 10 613 9 782 8 925 8 182 7 306 6 579 5 925 5 332 4 807	$\frac{16\,000}{1+\frac{l^2}{20\,000\,r^2}}$	Carnegie Standard.
Cast Iron: Round columns	$11\,300 - 30\frac{l}{r}$	10 000 reduced by Gordon's formulæ.	$\frac{11\,670}{1+\frac{l^2}{400\,d^2}}$	$\frac{l}{r} < 50$, 10 000 $\frac{l}{r} > 50$, $\frac{11\,000}{1+\frac{l^2}{10\,000\,r^2}}$	$\frac{l}{d} > 10$, $11\,100 - 220\frac{l}{d}$	$11\,300 - 30\frac{l}{r}$	$\frac{14\,000}{1+\frac{l^2}{600\,d^2}}$
Rectangular do	$11\,300 - 30\frac{l}{r}$	10 000 reduced by Gordon's formulæ.	$\frac{11\,670}{1+\frac{l^2}{400\,d^2}}$	$\frac{l}{r} < 50$, 10 000 $\frac{l}{r} > 50$, $\frac{11\,000}{1+\frac{l^2}{10\,000\,r^2}}$	$\frac{l}{d} > 10$, $11\,100 - 220\frac{l}{d}$	$11\,300 - 30\frac{l}{r}$	$\frac{14\,000}{1+\frac{l^2}{850\,d^2}}$

Material.	San Francisco, 1907.	Milwaukee, 1901.	District of Columbia, 1902.	Minneapolis, 1907.	Rochester, 1904.	New Haven, 1905.	Schneider's 1910.
Columns: Steel $\frac{l}{r}$ = 40 … 240	Carnegie Standard.	$\frac{15\,000}{1+\frac{l^2}{36\,000\,r^2}}$	$15\,200 - 58\frac{l}{r}$	$\frac{l}{r} < 90$, 12 000 $\frac{l}{r} > 90$, $17\,000 - 57\frac{l}{r}$	$15\,200 - 58\frac{l}{r}$	12 000 reduced by Gordon's formulæ.	$16\,000 - 70\frac{l_1}{r}$ 14 000 max.
Cast Iron: Round columns	$\frac{8\,000}{1+\frac{l^2}{800\,d^2}}$	$\frac{8\,000}{1+\frac{l^2}{800\,d^2}}$	$11\,300 - 30\frac{l}{r}$	$\frac{13\,300}{1+\frac{l^2}{400\,d^2}}$	$11\,300 - 30\frac{l}{r}$	13 333 reduced by Gordon's formulæ.	
Rectangular do	$\frac{8\,000}{1+\frac{l^2}{1\,670\,d^2}}$	$\frac{8\,000}{1+\frac{l^2}{800\,d^2}}$	$11\,300 - 30\frac{l}{r}$	$\frac{13\,300}{1+\frac{l^2}{500\,d^2}}$	$11\,300 - 30\frac{l}{r}$	13 333 reduced by Gordon's formulæ.	

TABLE 14 F (*continued*).

Permissible Unit Stresses—Iron, Steel, and Wood.

Oak, $\frac{l}{d}$ = 0–15 15–30 30–40 40–45 45–50	$900 - 17\frac{l}{d}$	750 650 560 460 375		$\frac{l}{d} < 12$, 1 000 $\frac{l}{d} > 12$, $1\,125 - \frac{125}{12}\frac{l}{d}$	$\frac{l}{d} < 10$, 800 $\frac{l}{d} > 10$, $980 - 18\frac{l}{d}$	$900 - 9\frac{l}{d}$	$\frac{l}{d} < 12$, 800 $\frac{l}{d} > 12$, $750 - 7.5\frac{l}{d}$			$900 - 17\frac{l}{d}$	$\frac{l}{d} < 12$, 800 $\frac{l}{d} > 12$, $750 - 7.5\frac{l}{d}$	$900 - 17\frac{l}{d}$	900 reduced by Gordon's formulæ.	$\frac{l}{d} < 10$, 1 000 $\frac{l}{d} > 10$, $1\,000 - 10\frac{l}{d}$
Yellow pine, $\frac{l}{d}$ = 0–15 15–30 30–40 40–45 45–50	$1\,900 - 18\frac{l}{d}$	1 000 875 750 625 500	$750 - 7.5\frac{l}{d}$	$\frac{l}{d} < 12$, 1 000 $\frac{l}{d} > 12$, $1\,125 - \frac{125}{12}\frac{l}{d}$	$\frac{l}{d} < 10$, 800 $\frac{l}{d} > 10$, $980 - 18\frac{l}{d}$	$1\,000 - 10\frac{l}{d}$	$\frac{l}{d} < 10$, 1 000 $\frac{l}{d} > 10$, $1\,000 - 10\frac{l}{d}$			$1\,000 - 18\frac{l}{d}$	$\frac{l}{d} < 12$, 1 000 $\frac{l}{d} > 12$, $1\,000 - 10\frac{l}{d}$	$1\,000 - 18\frac{l}{d}$	1 000 reduced by Gordon's formulæ.	$\frac{l}{d} < 10$, 1 000 $\frac{l}{d} > 10$, $1\,000 - 10\frac{l}{d}$
White pine, $\frac{l}{d}$ = 0–10 10–35 35–45 45–50	$800 - 15\frac{l}{d}$	625 475 375 300		$\frac{l}{d} < 12$, 800 $\frac{l}{d} > 12$, $925 - \frac{125}{12}\frac{l}{d}$	$\frac{l}{d} < 10$, 700 $\frac{l}{d} > 10$, $870 - 17\frac{l}{d}$	$700 - 7\frac{l}{d}$	$\frac{l}{d} < 12$, 700 $\frac{l}{d} > 12$, $625 - 6\frac{l}{d}$			$800 - 15\frac{l}{d}$	$\frac{l}{d} < 12$, 700 $\frac{l}{d} > 12$, $625 - 6\frac{l}{d}$	$800 - 15\frac{l}{d}$	600 reduced by Gordon's formulæ.	$\frac{l}{d} < 10$, 600 $\frac{l}{d} > 10$, $600 - 6\frac{l}{d}$
Spruce, $\frac{l}{d}$ = 0–10 10–35 35–45 45–50	$800 - 15\frac{l}{d}$	625 475 375 300	$500 - 5\frac{l}{d}$	$\frac{l}{d} < 12$, 800 $\frac{l}{d} > 12$, $925 - \frac{125}{12}\frac{l}{d}$		$700 - 7\frac{l}{d}$						$800 - 15\frac{l}{d}$	600 reduced by Gordon's formulæ.	$\frac{l}{d} < 10$, 600 $\frac{l}{d} > 10$, $600 - 6\frac{l}{d}$

TABLE 14 F (*continued*).

Permissible Unit Stresses—Iron, Steel, and Wood.

Material.	New York, 1906.	Chicago, 1905.	Philadelphia, 1907.	Baltimore, 1908.	St. Louis, 1907.	Boston, 1907.	Buffalo, 1905.	San Francisco, 1907.	Milwaukee, 1901.	Dist. of Columbia, 1902.	Minneapolis, 1907.	Rochester, 1904.	New Haven, 1905.	Schneider's, 1910.
	Pounds per Square Inch.													
Columns (continued): Locust				$\frac{l}{d} < 12$, 1 200; $\frac{l}{d} > 12$, $1\,325 - \frac{125}{12}\frac{l}{d}$						$1\,000 - 18\frac{l}{d}$		$1\,200 - 22.5\frac{l}{d}$		
Columns: Maximum $\frac{l}{r}$			140					Steel, 120		Steel, 120; Cast iron, 70		Steel, 120; Cast iron, 70		Steel, main membs., 125; Steel, secondary membs., 150
Maximum $\frac{l}{d}$	Steel, 40; Cast iron, 20	Cast iron, 24	45			Wood, 30	Steel, 40; Cast iron, 30	Cast Iron, 20		Wood, 30	Steel, 40; Cast iron, 30	Wood, 30		Wood, 45
Structural steel: Ultimate strength	54 000 to 64 000		Mild > 58 000; Medium > 65 000	60 000 to 70 000		55 000 to 65 000		60 000 to 70 000		54 000 to 64 000	> 54 000	54 000 to 64 000		55 000 to 65 000

TABLE 14 F (*continued*).

Permissible Unit Stresses—Iron, Steel, and Wood.

Shear:														
Steel	9 000	10 000	*10 000	9 000	12 000	10 000	10 000	9 000		9 000	7 000	9 000	10 000	10 000
" shop rivets and pins	10 000		11 000	10 000	†9 000	10 000	9 000	10 000	9 000	10 000	9 000	10 000	9 000	12 000
" field rivets	8 000		8 800	8 000	7 000		8 000	8 000	7 500	8 000	8 000	8 000	7 500	10 000
" " bolts	7 000			7 000	7 000	8 000				7 000		7 000		10 000
Cast iron	000			3 000						3 000		3 000		1 500
Yellow pine (with grain)	70	100	67	100		100				70		70		100
" " (across ")	500	250	750	500		500				500		500		
White " (with ")	40	80		85		80		100		40		40		100
" " (across ")	250	150		350		250		500		250		250		
Spruce (with ")	50	80	50	90		80				50		50		100
" (across ")	320	150	500	350		250				320		320		
Oak (with ")	100	150		100		150				100		100		200
" (across ")	600	250		720		600				600		600		
Locust (with ")	100									100		100		
" (across ")	720									720		720		
Hemlock (with ")	40		40	75								40		100
" (across ")	275		410	350								275		
Chestnut (across ")	150			150								150		
Bending:														
Rolled steel beams	16 000	16 000		16 000		16 000	16 000	16 000		16 000	16 000	13 000	16 000	16 000
Steel pins, rivets and bolts	20 000	22 500		20 000		22 500				20 000		20 000		24 000
Riveted steel girders, net sect.	14 000	15 000		15 000			13 500	15 000		14 000	13 500	14 000	12 500	16 000
Cast iron (tension)	3 000		3 750	5 000				3 000		3 000		3 000		2 500
" " (compression)	16 000			16 000				16 000		16 000		16 000		12 000
Yellow pine	1 200	1 250	1 600	1 800		1 500				1 200		1 200		1 500
White "	800	750		1 000		1 000		700		800		800		1 000
Spruce	800		1 100	1 350		1 000				800		800		1 000
Oak	1 000	1 000		1 500		1 000				1 000		1 000		1 200
Locust	1 200									1 200		1 200		
Hemlock	600		900	1 000								600		800
Chestnut	800									800		800		

* Soft steel = 8 750. † For Rivets only. Pins = 12 000.

TABLE 14 G.

Permissible Unit Stresses—Concrete and Reinforced Concrete.

Pounds per Square Inch.

	Chicago, 1905.	Philadelphia, 1907.	Baltimore, 1908.	St. Louis, 1907.	Boston, 1907.	Buffalo, 1905.	San Francisco, 1907.	Minneapolis, 1907.	Schneider's, 1910.
Concrete:									For 2 000 lb. concrete.
Compression: Direct	......	Stone ...500 Slag ....300 Cinder..150	500	Burnt clay..300 All other....500	350	350	450	350	Col's......400 Bear'g ... 600
Compression: Fiber strain	......	Stone ...600 Slag ....400 Cinder..250	500	Burnt clay..400 All other...800	1:5, 500 1: < 5 600	500	500	500	500
Compression: Spiral winding or hooping cols.	......	Stone. 1 000	1 200	Special.	......	......	700	......	Tubes 700
Shear	......	Stone....75 Slag......50 Cinder...25	50	......	60	50	75	50	60
Bond	......	Stone....50 Slag......40 Cinder...15	60*	Medium steel, 50, high elas. lim. steel, 30	60	50	75*	Bars ¾" ϕ, 75 " > ¾" ϕ, $75 \times \frac{0.75}{\text{diam.}}$	60
Bond to deformed bars	......	......	......	By test.	......	......	......	100	......
Steel:									
Tension	......	......	......	14 000 & 20 000	16 000	16 000	......	16 000	16 000
Compression	......	......	......	......	16 000	......	......	12 000	16 000
Shear	......	16 000	8 000 & 10 000	......	10 000	10 000	10 000	10 000	......
Fiber Strain: Tension	......	16 000	12 000 & 15 000	14 000 & 20 000	16 000	......	⅓ Elastic limit.	......	......
Fiber Strain: Compression	......	16 000	8 000 & 9 000	14 000 & 20 000	16 000	......	......	......	......

* Steel with an elastic limit > 40 000 lb. shall have mechanical bond.

TABLE 14 H.

Further Requirements—Concrete and Reinforced Concrete.

	Chicago, 1905.	Philadelphia, 1907.	Baltimore, 1908.	St. Louis, 1907.	Boston, 1907.	Buffalo, 1905.	San Francisco, 1907.	Minneapolis, 1907.	Schneider's 1910.
Mixture of concrete	1:3:5	1:2:4	1:2:4	1:2:4	1:5	....	1:6	1:2:4	1:6
Compressive strength of concrete after 28 days (lb. per sq. in.)	2 000	....	....	Burnt clay, 1 000 All other, 2 000	2 000	2 000	2 000	2 000	2 000
Ratio of moduli of elasticity of concrete and steel	....	Stone 1:12 Slag 1:15 Cinder 1:30	1:15	Burnt clay, 1:20 All other, 1:15	For beams and slabs, 1:15 for cols. 1:10	1:12	1:15	....	Stone, 1:15 Cinder, 1:30
Columns: Maximum ratio of height to least lateral dimension	12	15; if > 15 reduce unit str.	16	15	120 r	16	15	12	12
Slabs: Bending moment, continuous	....	$\frac{Wl}{10}$; at wall $\frac{Wl}{8}$	$\frac{Wl}{10}$; at wall $\frac{Wl}{9}$	$\frac{Wl}{12}$	$\frac{Wl}{10}$	$\frac{Wl}{10}$	$\frac{Wl}{12}$	....	$\frac{Wl}{10}$
Bending moment, square and supported on four sides	....	$\frac{Wl}{20}$; at wall $\frac{Wl}{16}$	....	....	....	$\frac{Wl}{20}$	$\frac{Wl}{20}$	$\frac{Wl}{20}$	$\frac{Wl}{20}$
Beams: Width of slab allowed for T bm. action	....	20 × thk's of slab	4 × width of beam	$\frac{1}{4}$ span	$\frac{1}{3}$ span	10 × width of beam	5 × thk's of slab	....	8 × thkn's of slab or $\frac{1}{3}$ span
Bending moment, continuous	......	$\frac{Wl}{8}$	$\frac{Wl}{10}$; at wall $\frac{Wl}{9}$	$\frac{Wl}{8}$	$\frac{Wl}{10}$	$\frac{Wl}{8}$	$\frac{Wl}{8}$	$\frac{Wl}{8}$	$\frac{Wl}{10}$
Tests: Load	....	2 × safe load	....	2 × safe load	....	3 × live load	2 × safe load	2 × safe load.	2 × live load.
Deflection allowed	....	No sign of crack	....	....	$\frac{1}{360}$ span	....	$\frac{1}{700}$ span	Beams $\frac{1}{1000}$ span. Slabs $\frac{1}{300}$ span	No permanent deformation.

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